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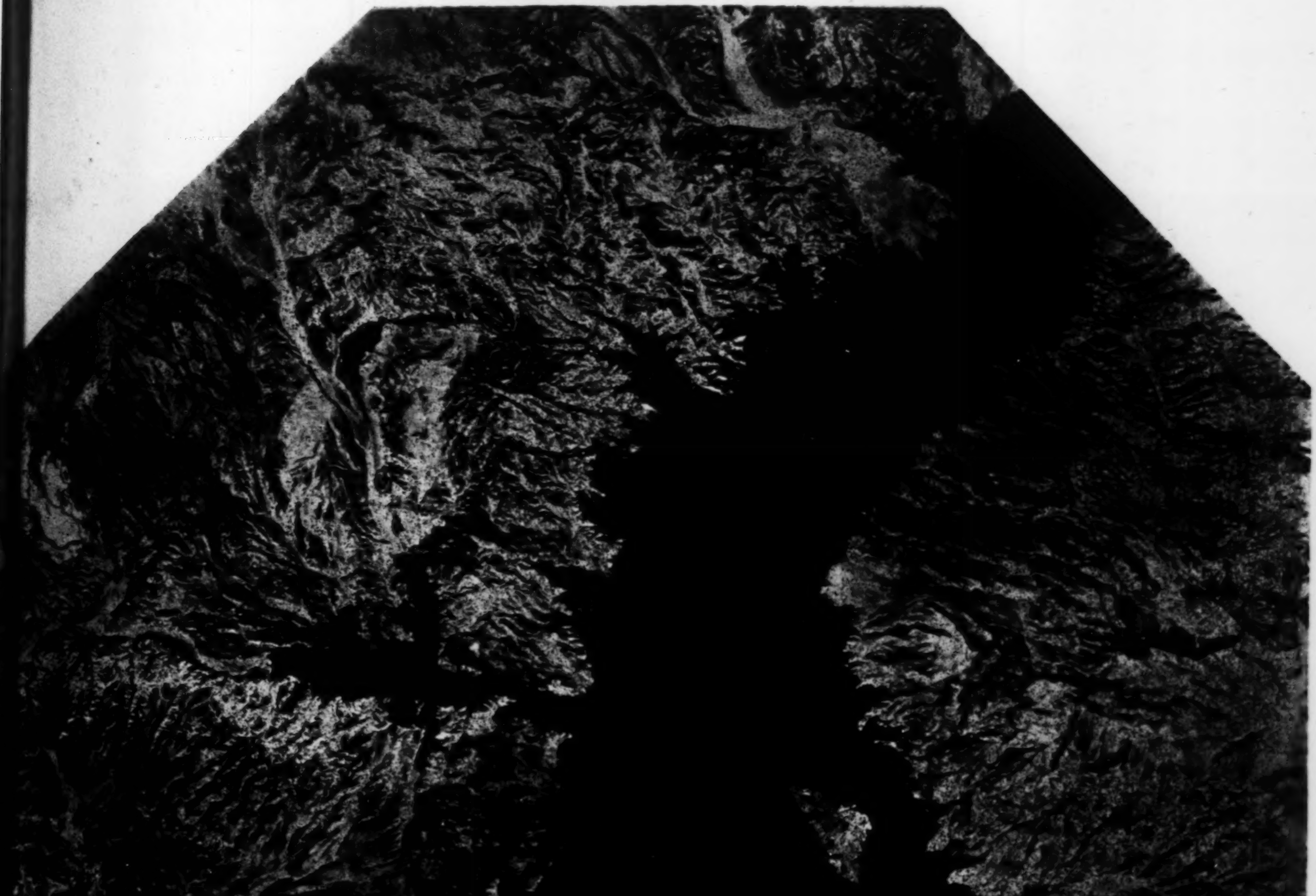

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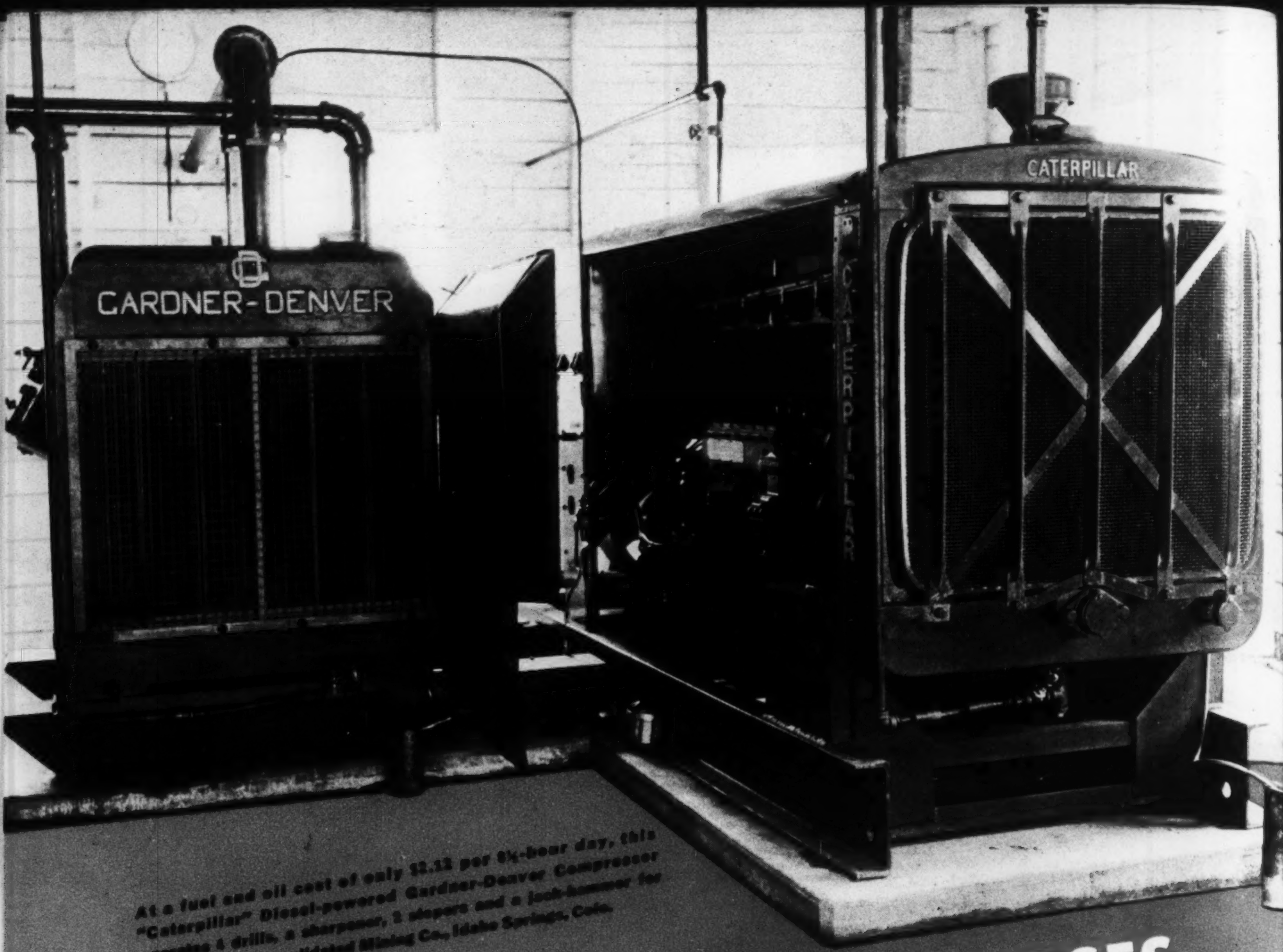
From 20,000 feet up Boulder Dam for once does not dominate its surroundings, showing only as a thin crescent at the lower right of this view of the huge lake that it is impounding.



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DURING the last few years construction has loomed large in the news. This is but natural in view of the far-reaching effects upon general business of the drop in private construction and of the efforts to stimulate public construction as a recovery measure.

Because of this heightened interest of the general public, many newspapers have sought to inform themselves more fully concerning the operations of the industry; it has been noteworthy that editorial comment on the subject has been sound and sensible just about in proportion as editorial writers have taken competent counsel with respect to it.

In this connection we frequently are asked by newspaper editors about the significance of this, that or the other development that involves the construction industries. More often such requests come from readers who seek permission to reprint or otherwise to circulate for general distribution some article or editorial in our columns that may have a special bearing on the public interest.

NOW comes a letter from a valued subscriber which will interest many readers as indicating how they can help to get general circulation for material that may help to effect a more widespread understanding of the special problems that beset the industry. It reads as follows:

> "A short time ago I submitted to the editorial writer of ——, one of the leading newspapers of this part of Ohio, one of the *Engineering News-Record* editorial pages. I am enclosing an editorial based on one of yours that they published in their issue of June 28. I thought you might be interested in this, as the paper reaches thousands of readers who, of course, never see or hear of *Engineering News-Record.*
>
> "I do not know whether or not your editorials are placed in the hands of any of the leading newspapers of the country, but I am satisfied that if they should be, many of them would be willing to use them. I do this myself when an editorial appears which I feel the newspapers would be willing to handle. If there is any way of having this done throughout the country I believe that it would be quite effective."

The practice of this reader and of others who from time to time report similar action is commended to those who wish to help in their own communities toward a wider understanding of construction. Needless to say, such contact will be most effective when the man who forwards the article has a personal acquaintance with the editor or someone on his staff; and to gain such acquaintance should not be difficult for the responsible citizen. Regardless of the particular form of cooperation, engineering and construction would be much better off in every community if its leaders would maintain consistently a close contact with their local newspapers. The right-minded editor welcomes such cooperation from qualified citizens so long as it is directed toward more intelligent handling of public affairs rather than the promotion of special interest. Certainly such contacts will help insure more accurate news of the industry and to keep editorial judgment more aware of its special problems.

The staff of *Engineering News-Record* tries to bring such of its content as may be of general interest to the attention of newspapers the country over; their frequent use of such material shows that this cooperation is welcome and fruitful. But such general effort cannot be so effective as personal suggestion by local citizens, known to the editors not only for their standing in their own industry but also for their responsibility as members of their communities.

THE statement often is made that the public has less understanding and appreciation of the construction industries than of other business. Whether or not this be true there surely is room for improvement. The technical and industrial journal must continue to do its part, under its growing responsibility to interpret the industry to the outside world as well as to serve it directly; but this does not decrease the responsibility of each individual to keep his local newspapers alive to the needs and the problems of his industry.

Our Ohio reader is doing a good job for his profession and his industry; many engineers, architects, constructors, manufacturers and distributors will thank him for a suggestion as to how they can help in their own communities.

Willard Chevalier

ENGINEERING NEWS-RECORD

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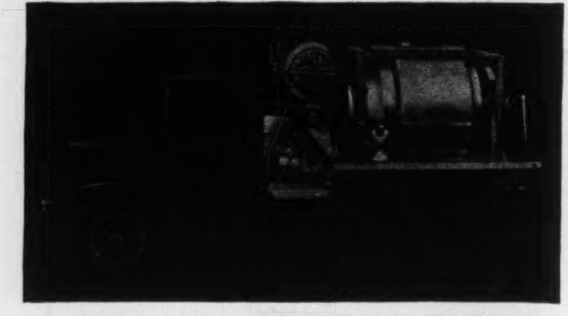

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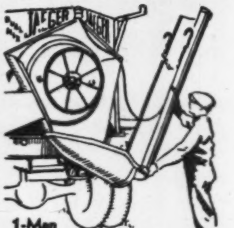

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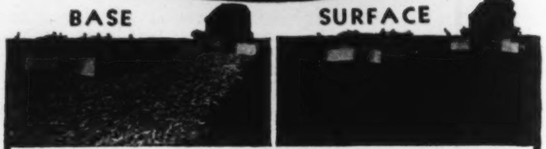

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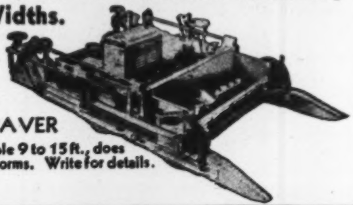

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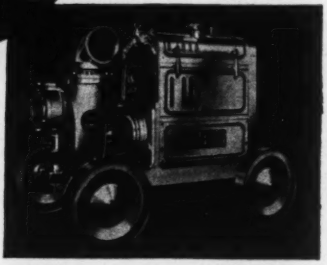

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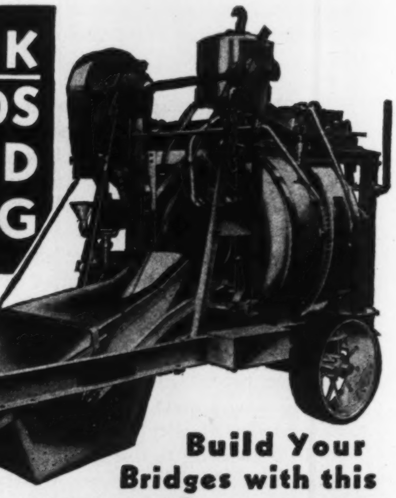

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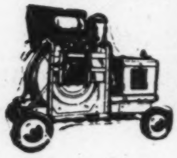

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In the News:

THE "ROAD SHOW" is to be revived. The American Road Builders Association has announced that after a lapse of two years there will be a road show in connection with the annual convention at Cleveland, Jan. 20-24. The show is to be in the city's new exhibition hall.

TWO IMPORTANT BILLS, clarifying the federal government's authority over large construction enterprises, were signed by the President last week. The Rivers and Harbors Act permits resumption of work on Parker Dam on the Colorado River, the battleground of Arizona's "Navy," and also specifically authorizes the construction of Grand Coulee Dam in Washington, which has been under construction for many months. The second bill signed was one strengthening the powers of the Tennessee Valley Authority.

TVA WAS IN THE NEWS for another reason also. This was its formal adoption of a liberal labor policy covering rates of pay, hours of work, collective bargaining and other labor standards for the benefit of its 17,000 employees.

WPA FUNDS are available in New York City for new roadways on the Williamsburg suspension bridge (nearly $2,000,000) and the Queensboro cantilever bridge ($750,000), both over the East River.

AN OMNIBUS BRIDGE BILL authorizing or extending the time for construction of 39 bridges, private and public, in various parts of the country, was signed by the President on Aug. 31.

NEW YORK CITY began demolition work last week precedent to construction of a first link in the long-projected East River Drive, to run the length of Manhattan Island. This first section runs from Grand to 14th St. Farther uptown the Triborough Bridge Authority is also building a section from 92d to 122d St., as an approach to its bridge.

THE ETHIOPIAN SITUATION has taken on a new interest for engineers with the announcement that construction of a dam to control Lake Tsana, headwaters of the Blue Nile, was included in one of the sensational concessions made last week by the king of Ethiopia. Though the concession relating to Lake Tsana is reported to have been granted to a British concern, it was an American firm, the J. G. White Engineering Co., that made reconnaissance studies of the project for the king. That company's parties, under L. B. Roberts, who writes on Ethiopia in the current *National Geographic*, made two surveys for a possible highway across the difficult mountainous region between Addis Ababa and the lake, as well as a survey of the lake itself.

In This Issue:

TIMBER CRIBS for the cofferdams of Bonneville Dam on the Columbia River were built in 1,400-ton units and floated into line in a seven-mile current by means of ingenious cable control. The placing, sinking in 50 ft. of water and dewatering of these cofferdams comprised a construction operation of unusual character.

THE SCIENCE OF SOIL MECHANICS is gradually being built up on a foundation of new theories discovered and studied in the engineering laboratories. From Professor Casagrande at Harvard come new data on consolidation characteristics of clays and the behavior of soils subjected to shearing stresses.

A CONCRETE THROUGH-ARCH BRIDGE with the record-breaking span of 528 ft. has been built in France. Hollow ribs are used, and in other particulars the structure is noteworthy.

EXTRAORDINARY subsurface conditions not disclosed by borings caused a new earth dam for the Reading, Pa., water department to cost 2½ times the amount of the original bid. The difficulties of construction originated in a badly eroded limestone bed that followed the line of the corewall, requiring extensive drilling and grouting.

IN HIS THIRD ARTICLE on "Early Days at Panama," John F. Stevens describes further the contributions made by civilian engineers to the construction of the canal.

A LABORATORY INVESTIGATION of steel beam-to-column connections with respect to reduced bending moments in the beams caused by the restraints of their ends was carried out at Worcester Polytechnic Institute. In addition to the interesting procedure utilized by the investigators, the data that they secured on a question whose importance is not widely appreciated commend the article for attention.

Coming Articles:

THE CAUSE AND PREVENTION of transverse fissures in rails have been subjects of deep concern to railroads for many years because of the large number of rail failures in service due to such fissures. In 1931, rail manufacturers and the railroads joined in a study that has cleared up many of the questions involved and points definitely to methods by which the formation of transverse fissures can be greatly reduced if not entirely eliminated. The first information concerning that study is contained in a forthcoming bulletin of the American Railway Engineering Association. An abstract of that bulletin, prepared by Prof. H. F. Moore who was in charge of the test party, will be published in the issue of Sept. 19.

THE TIME FOR "DIAGNOSIS"

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the rope
is used

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September 5, 1935

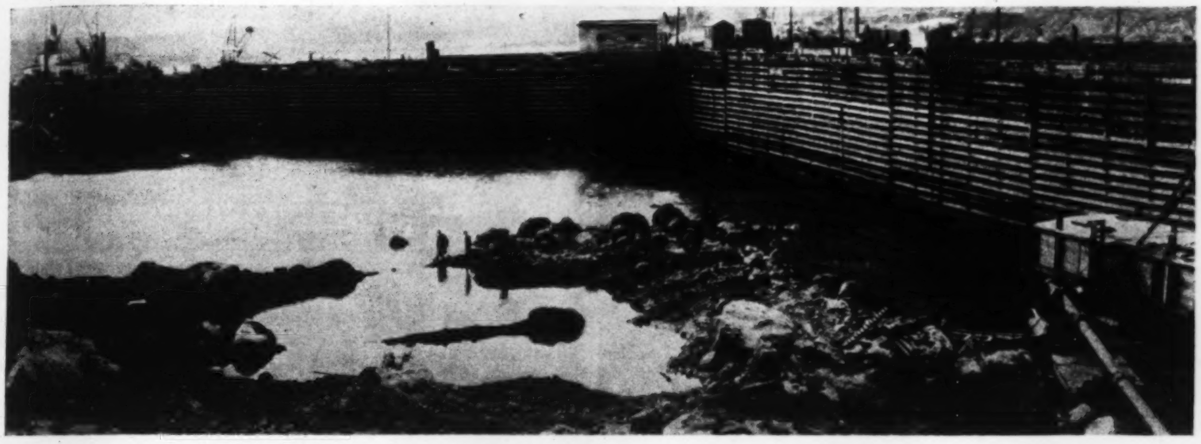


FIG. 1—INSIDE of cofferdam at Bonneville, showing crib construction and character of unwatered riverbed before excavation for the dam foundations.

Cofferdams in Swift Water for Bonneville Dam

Floating 1,400-ton cribs into line in a seven-mile current by cable control in constructing south cofferdam for the Bonneville Dam on the Columbia River

By C. I. Grimm
Chief Civilian Engineer, Bonneville Project, U. S. Engineer Office, Portland, Ore.

COFFERDAMMING presents a major design and construction problem in building the Bonneville Dam on the Columbia River. A channel 1,000 ft. wide has to be progressively laid dry for building the concrete spillway dam 1,250 ft. long and 170 ft. high. This work has to be done in water from 30 to 50 ft. deep, in currents up to 7 miles an hour and on a rough and irregular riverbed. Moreover, the construction has to meet the condition that during the flood season of the river the works would be inundated. Planning and construction of unusual character for river closure were necessary to meet these trying requirements, and they are outlined in this article. General descriptions of the project as a whole and of the progress of construction have been published in *Engineering News-Record,* Nov. 1 and 29, 1934, and July 4, 1935.

Plan and progress

The plan of closure selected was to unwater the foundations in two successive steps. First a cofferdam was built for the south or Oregon half of the dam; after the concrete dam foundations have been poured on this side, the river will be turned to the south half of the channel in a low-water season while the Washington half of the site is similarly unwatered.

Low-water season in the Columbia comes in winter when much of the drainage basin contributes but little water. Beginning in April, melting snows and late rains combine to bring high water that lasts usually until August.

During the low-water season of 1934-1935 the first half of the cofferdam was constructed. This involved building 21 timber cribs on shore one at a time; after launching they were towed to place and sunk. This was a difficult task in water 30 to 50 ft. deep, with a current of 7 m.p.h., but it was successfully completed; and after sealing and unwatering the cofferdam, almost 200,000 cu.yd. of river fill and rock were excavated from the 8-acre area within it. Just before the arrival of high water all excavating and pumping equipment was removed, the cofferdam was allowed to fill gradually, and summer floods overflowed the tops of the cribs. This is in accord with the construction plan. During the first half of the present low-water season (beginning in late August) the cofferdam is to be pumped out, excavation completed and the concrete foundation placed. During the second half of the low-water period the cofferdam will be removed and a similar one will be constructed in the north half of the channel.

Design and construction

Because of the special difficulty and risk involved in the cofferdamming operation at Bonneville and also because of the limited number of contractors prepared to assume such a risk, it was early decided that it would be advisable for the United States to undertake this responsibility. The specifications were drawn accordingly, so that the cofferdams are designed, built and inspected in a manner very similar to those employed for the permanent structure. The excavations for crib foundation, timber, fill, steel piling, pumping, etc., were specified and bid on a unit-price basis, and the coffers are being built to designs made by the engineers. Detail construction drawings are furnished the con-

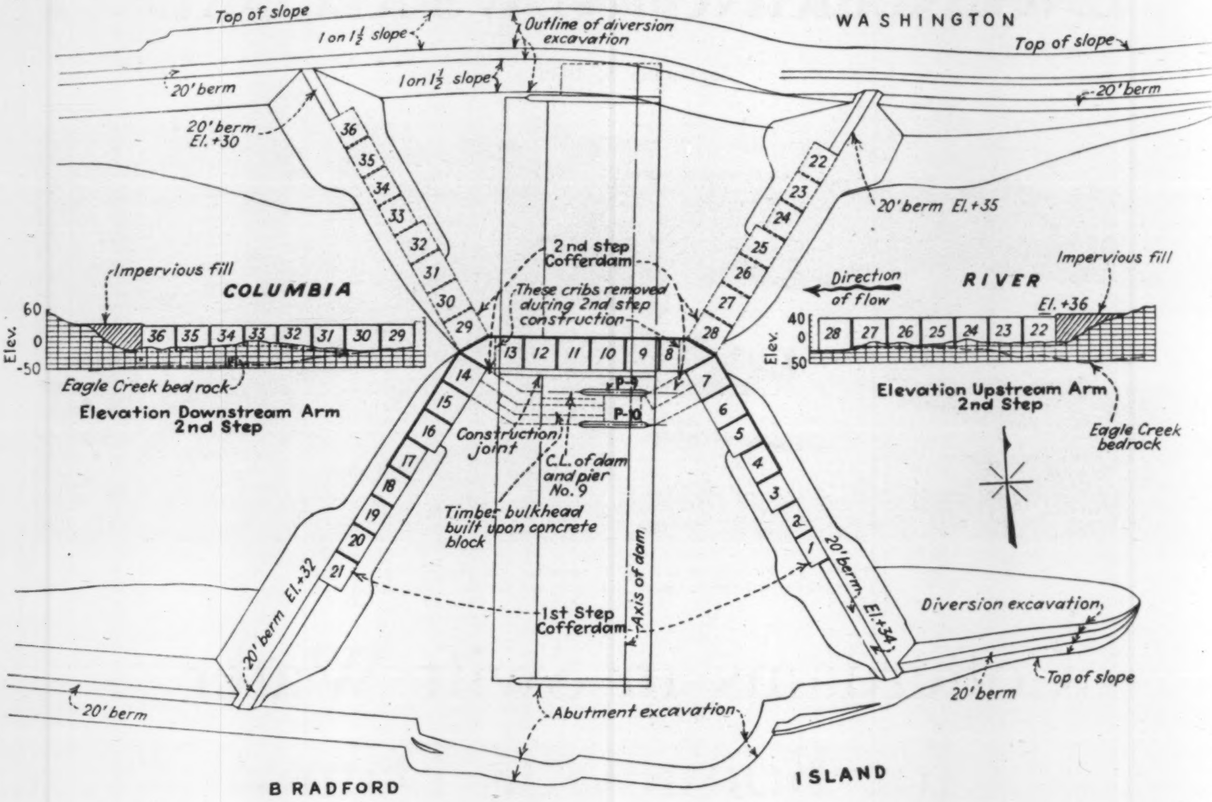

WASHINGTON
Top of slope
1 on 1½ slope
Outline of diversion excavation
1 on 1½ slope
20' berm
20' berm El. +30
Top of slope
20' berm
20' berm El. +35
2nd step Cofferdam
These cribs removed during 2nd step construction
Impervious fill
COLUMBIA
RIVER
Direction of flow
Impervious fill
El. +36
Elev.
60
0
-50
36 35 34 33 32 31 30 29
Eagle Creek bed rock
Elevation Downstream Arm 2nd Step
40
0
-50
28 27 26 25 24 23 22
Elevation Upstream Arm 2nd Step
Eagle Creek bedrock
13 12 11 10 9 8
P-9
P-10
14 15 16 17 18 19 20 21
7 6 5 4 3 2 1
Construction joint
C.L. of dam and pier No. 9
Timber bulkhead built upon concrete block
Axis of dam
1st Step Cofferdam
20' berm El. +32
20' berm El. +34
Diversion excavation
Top of slope
20' berm
20' berm
Abutment excavation
BRADFORD
ISLAND

FIG. 2—PLAN of south cofferdam at Bonneville, with tentative plan of north cofferdam indicated by dash lines to show overlap at center of channel.

tractor for each crib, including the timber, bolting and layout of the bottom to fit soundings made by the engineers. The contractor is responsible for the proper placing and loading of the cribs, but after a crib has been built to specifications, acceptably placed and loaded, the United States assumes responsibility for its stability when unwatered or its destruction or loss by flood, ice or any other natural cause. In case of such loss, the United States will pay for the reconstruction of the cofferdam and any of the enclosed works that are damaged, at the bid unit price, and will extend the contractor's time of completion to compensate for the time lost.

This action having been determined, the entire cofferdam and river-diversion plan was tested by a hydraulic model of 1:100 ratio. The experiments related to the division of flow between channels; to the slopes, direction and velocities of current; to the tendency to scour river banks and bottom; to the handling of cribs; and to the order of placing the cribs, etc. They were found to be very useful. Experience on the prototype to date has proved the reliability of the model tests.

Considerations that led to the use of shore arms inclined in plan (Fig. 2) were: (1) that this shape offered nearly a minimum obstruction to river flow for stages either below or above the top of the cofferdam and did not produce eddies or cross-currents tending to scour the river banks; and (2) that a greater width of cofferdam was required at the river bank for the construction of upstream and downstream abutment walls and fishways.

The cribs were designed for safety against sliding and overturning, taking into account the nature of the foundation and the material used for filling. Detailed calculations for several heights showed that the simple rule of making the width equal to the height was applicable for cribs not supported on the inside by banking. Vertical members securely bolted at the crib corners and the corners of the weight pockets were used to strengthen the cribs against horizontal shear, to give them the rigidity necessary for handling in swift water, and to prevent parting along a horizontal plane while weighting and sinking. Bursting pressure exerted by the crib fill was found to be an important factor. The correctness of assumptions for this pressure was checked by embedding Goldbeck cells in one of the cribs. These data made available for the design of the cribs for the north half of the channel agreed reasonably well with assumed pressures (Fig. 3).

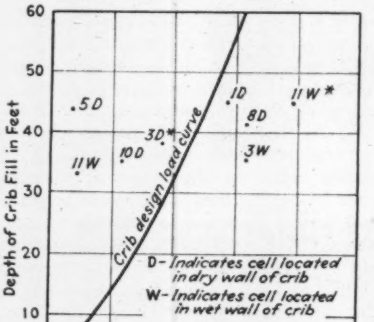


FIG. 3—CHART of assumed lateral pressures of fill in cofferdam cells and pressures as determined by Goldbeck cells.

Various types of full-size bolted crib joints were tested to determine allowable design loads. These tests gave the ultimate shearing strength of drift-bolted joints as about 5,000 lb. per ¾x22-in. bolt and showed that a working stress of 2,000 lb. per bolt could be used without material yielding. Split rings of the type now commonly used in wood-truss and tower construction were found to be economical at highly stressed joints. Fig. 4 shows details of the crib structures; at first it was thought to build them for a developed bottom, but this plan was revised after taking soundings.

In the swift deep water it was found that soundings made with a wire line and heavy lead weight did not develop with reasonable accuracy the very irregular bottom at the cofferdam site, and a special sounding rig was constructed. This consisted essentially of four vertical pipe wells rigidly connected to the downstream end of a steel barge and spaced 4 ft. center to center. These pipes reached to within 10 ft. of the average river bottom, and soundings through them were made by means of steel rods, the barge being held in place and shifted by means of upstream and side lines. Soundings were ordinarily taken 4 ft. center to center in both directions.

After the first soundings the crib foundation area was dredged by a 5-cu.yd. dipper dredge, with a view to preparing a more regular foundation and seating the cribs six to sixteen deep on bedrock overlaid in this area by 5 to 20 ft. of gravel and boulders, many of the boulders being greater than 4 ft. in diameter. Although dredging was carried to anticipated depths, the bottom when sounded afterward was found to be very irregular as a result of bedrock pinnacles and the boulders were too large to be moved by the dredge. The idea of smoothing the bottom was then abandoned as impracticable, and it was decided to construct the cribs so as to fit approximately the irregular bottom.

Launching and placing

The cribs were built up to a height of about 15 ft. on sloping ways on the river bank and were launched and moved to their position, where additional courses were added until the height exceeded by a few feet the depth from foundation to water surface. They were then sunk by dumping a mixture of gravel and boulders into the weight pockets. As soon as they touched bottom, soundings were again made from the crib top inside of all open pockets to determine if they were properly seated, and divers were sent down inside the open pockets to check the fit of the crib to the river bottom and to locate points of contact. It was necessary in a few cases to blast out boulders that prevented proper seating, and in one case some timbers were blown out to permit the crib to settle. In general, cribs landed within a foot of their theoretical horizontal position with tops fairly level and within a foot of their theoretical level. Accuracy in alignment and level is illustrated by Fig. 1. The cribs were placed in the following order: 13, 12, 11, 1, 10, 2, 3, 9, 8, 4, 7, 21, 6, 20, 5, 19, 14, 18, 15, 17, 16. (See Fig. 2.)

The handling of these large cribs in swift water developed a total pull on the upstream lines of as much as 175 tons and required carefully planned upstream and side lines and powerful hoists, which are illustrated by the accompanying diagram, Fig. 5. Actual pull on the lines was measured by the contractor with a tension meter and was found to agree closely with the theoretical pull as estimated from velocity of current and submerged area of crib. In general, the following formula was found to apply to pull on upstream lines: $P = 1.5 \times 62.5\ Ah_v$, where P = total pull on crib in lb.; 1.5 = a constant, depending on shape of crib; 62.5 = density of water in lb. per cu.ft.; A = gross and area of crib presented to current in sq.ft.; and h_v = velocity head of current in ft.

Material for filling crib No. 13 (the first one placed) was trucked from shore over a wooden-pile trestle constructed in the swift water to the site of this crib. Although steel pile points were used, very limited penetration was possible and the trestle was therefore well braced and anchored at the end with lines. Filling operations were then extended upstream over the cribs as constructed. Filling operations were also extended out from shore as the construction of the shore arms progressed riverward. The total quantity of crib fill was 114,000 cu.yd., about 4,000 cu.yd. of which was placed in sinking the cribs.

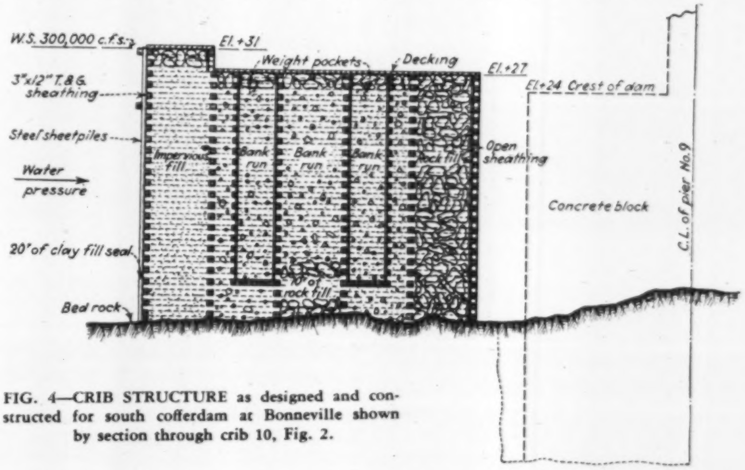


FIG. 4—CRIB STRUCTURE as designed and constructed for south cofferdam at Bonneville shown by section through crib 10, Fig. 2.

Sealing cribs

For watertightness, interlocking steel sheetpiles were driven along the side of cribs 4 to 17, inclusive, which were built with 3-in. tongue-and-groove sheathing on that side. Tongue-and-groove wooden sheathing and impervious banking were considered adequate on the river side of the other cribs. Along cribs 6 to 17, inclusive, the steel piles are driven into the bedrock; the piles penetrate the gravel and boulder formation under cribs 4 and 5. The space between the steel piling, which is the arch-web type, and the wood sheathing on the river side of the cribs was filled to a height of about 20 ft. above the river bottom, with clay as a further seal against leakage through the pile interlocks. Where space between cribs was not closed by steel piling, Wakefield piling was driven to close the gap in the tongue-and-groove crib sheathing. The average penetration into the relatively soft bedrock of about 2 ft. is indicated by the accurate driving-penetration records kept for this purpose as well as by the results of test driving into a similar rock on the south river bank. An inspection of all steel piles after driving was also made at the bottom of the cribs by a diver who descended into the outside row of crib pockets kept open for this purpose until the piles were driven and inspected. Tee-piles were placed at 12-ft. intervals in the pile wall, to permit driving a loop around leaks that might develop as a result of faulty penetration of the main pile wall. However, such leaks did not develop. The earthfills connecting the cribs with the bank were sealed by an impervious core extending into a dredged trench underneath and well into the riverbank.

It has been demonstrated in numerous cofferdams that the leakage through a properly constructed steel sheetpile wall is small. There was, however, the problem of preventing serious leakage through possible porous strata under the coffer or at the shore ends—particularly since the formation contains boulders of such size and extent that steel piling will not penetrate to any great extent. To design a cofferdam that could without doubt be unwatered and to estimate the pumping capacity required, extensive studies were made of the material through which seepage might occur. The riverbed material under the shore arms was tested by drill holes, and that in the banks by drill holes and open test pits. The pits were carried well below the adjacent riverbed, and the rate of inflow was measured by pumps. These explorations indicated material of such nature that leakage would not be serious. The total leakage into the cofferdam was estimated by the writer at 50 to 100 sec.-ft.

Unwatering and seepage

The cofferdam was unwatered by electrically driven, vertical shaft pumps.

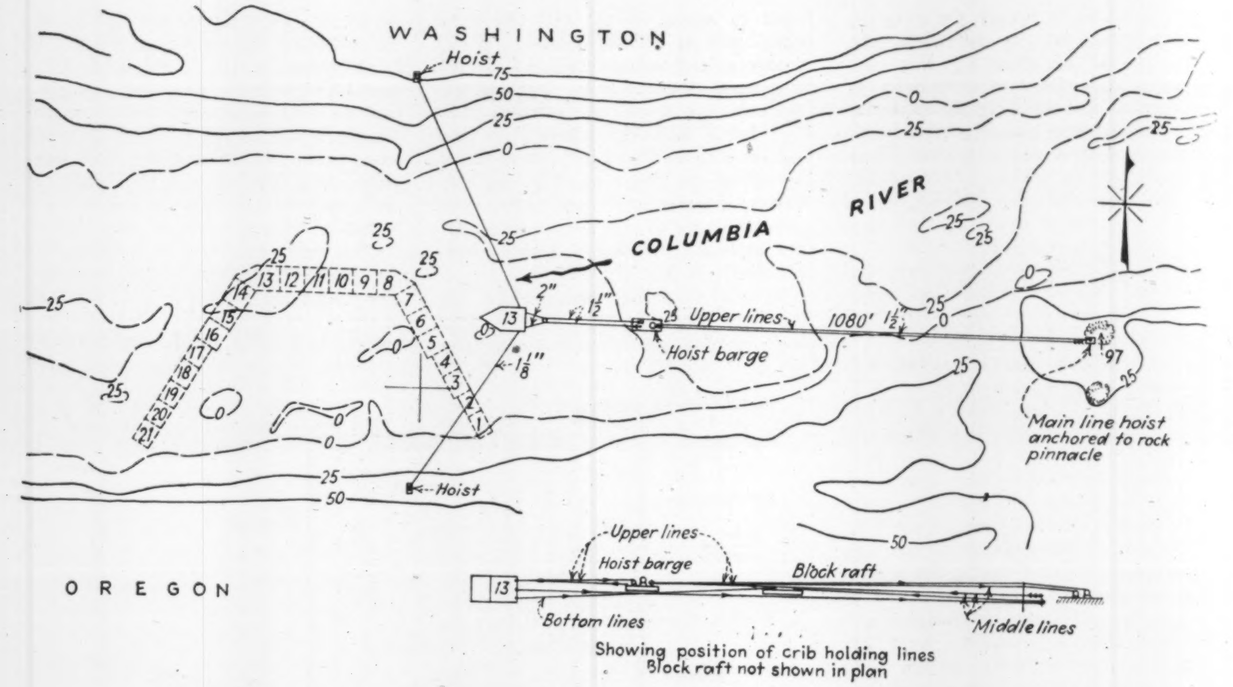


FIG. 5—HOLDING LINES and hoists as arranged for floating crib 13 into line for cofferdam wall.

Seven pumps having a combined capacity of 150 sec.-ft. were installed by the contractor on top of the cribs, and two pumps having a combined capacity of 20 sec.-ft. were mounted on a barge. After the river bottom was uncovered and excavation begun, some of the pumps were placed inside the cofferdam so as to reach the deeper pits. To avoid danger from blowouts, the rate of lowering the water within was controlled and the sealing of leaks was carried on as the pool was lowered.

Leakage through the steel piling was large at first but decreased after a head of about 15 ft. was created and the pressure had caused the cribs to deflect and the interlocks to tighten. The interlocks were further sealed by dumping silt and clay along the sloping upstream arm. This material was carried in suspension along the river face of the cofferdam, its movement being relatively slow by reason of the numerous eddies caused by the rough steel pile wall, and was drawn into joints of the piling.

TABLE I—SEEPAGE RECORDS FOR BONNEVILLE SOUTH COFFERDAM

Date	Gage Outside	Gage Inside	Head	Sec.-Ft. Inflow
Mar. 4	17.3	+13.7	3.6	125
" 5	17.3	+10.0	7.3	132
" 6	17.4	+ 5.0	12.4	145
" 7	17.3	0.0	17.3	124
" 9	17.1	— 5.0	22.1	98
" 9	17.0	—10.0	27.1	91
" 13	17.2	—15.0	32.2	83
" 15	17.1	—17.4	34.5	72
" 18	20.1	—14.0	34.1	92
" 20	19.4	—17.8	37.2	81
" 21	18.8	—16.1	34.9	76
" 25	18.5	—16.0	34.5	67
" 27	18.0	—16.0	34.0	61
" 31	....	......	....	61

TABLE II—MOVEMENT OF CRIB TOPS AT BONNEVILLE DAM AFTER UNWATERING

Crib No.	Width, Ft.	Aver. Height, Ft.	Seated on	Maximum Deflection, In.
2	36	44	Gravel	2*
4	48	47	Gravel	11
6	60	56	Rock	19
9	60	57	Rock	12
12	60	55	Rock	12
15	60	53	Rock	18
18	48	45	Gravel	10
20	48	45	Gravel	4*

*Supported by inside banking.

Fine cinders were then used, further to seal the interlocks where leaks were observed. The cinders were introduced below the water surface through a 3-in. pipe. Steel-pile interlocks above the lowest water level during construction were painted with an asphalt emulsion mixed with portland cement.

Some seepage around the shore ends of the cofferdams was materially reduced by dumping silt and clay in the water where seepage entered. Some rates of inflow during and after unwatering of the cofferdam are given in Table I.

Deflection of cribs

As the cofferdam was unwatered, the crib positions were checked to detect movement, particularly any sliding on the foundations. No evidence of sliding was found. Appreciable horizontal movement at the crib tops was measured as well as slight settlement of the crib tops on the dry side. Observed lateral movement at the tops of cribs increased as the water level within the cofferdam was lowered, and there was some further movement as a result of the shrinkage of timbers on the dry side from drying out.

For several typical cases the extent of maximum lateral movement at the crib tops, which varied with their height and foundations, is shown in the figures of Table II.

Contract cost

The contract cost to the government of constructing the cofferdam in the south half of the channel was about $1,000,000, of which the principal items were as follows:

Crib footing excavation	$223,000
Crib timber	530,000
Steel piles	167,000
Crib fill	86,000
Shore and toe fill	31,000
Total	$1,037,000

Engineering supervision

The general plans for the cofferdams and river diversion were developed by the writer, and detail designs of structures and construction drawings were prepared by H. G. Gerdes, designing engineer for the spillway dam, under the supervision of the chief engineer. F. H. Cothran, who has had wide experience on similar work, is consulting engineer for the government on the cofferdam and river-diversion features of the project.

The Bonneville project is under the supervision of the Corps of Engineers, U. S. Army. Col. T. M. Robins is division engineer, and Major Charles F. Williams is district engineer, at Portland, Ore.

Safety Feature in Earth Dam Spillway Tested in New York Flood

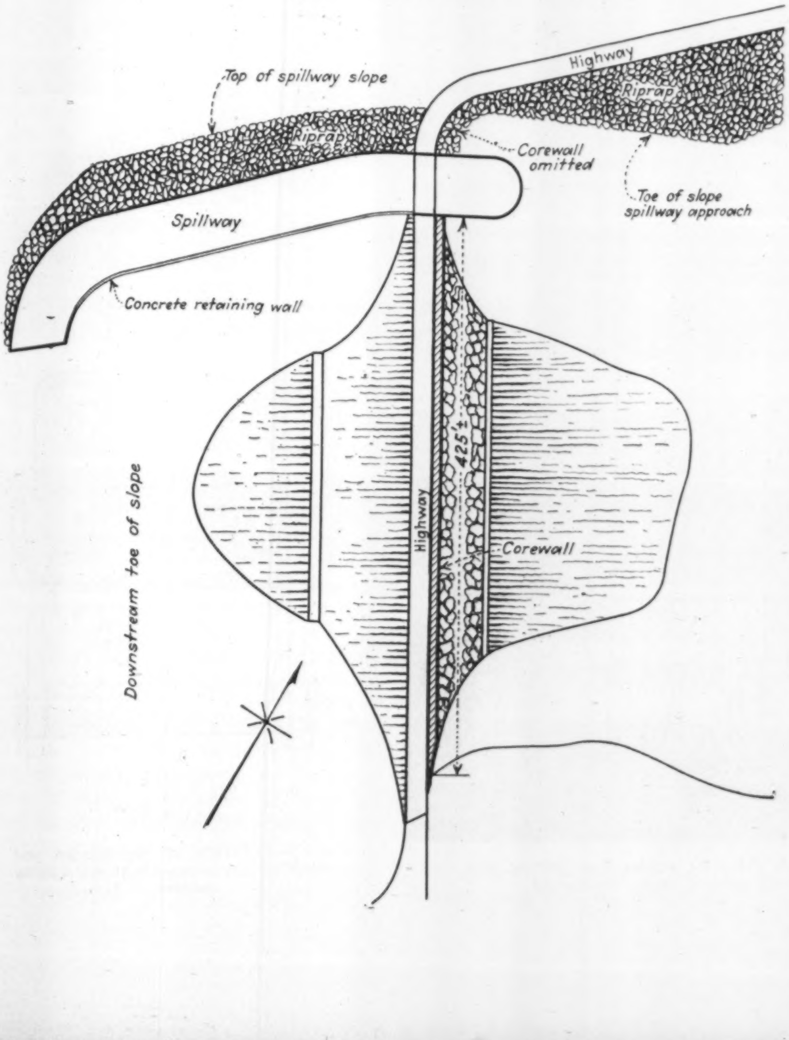
Failure of sections of riprap bears out design predictions when a head of 4.25 ft. over the spillway closely approaches the estimated 500-year possibility of a 5-ft. crest

By Wallace V. R. Fretts
Fretts, Tallamy & Senior, Consulting Engineers, Williamsville, N. Y.

RECENT FLOODS in central New York, which caused the destruction of one earthfill dam of the Hornell, N. Y., water-supply reservoir system, direct attention to the interesting safety device incorporated in another water-supply dam of the same system. This structure, built in 1931, was so designed by the writer's firm that previous to any possible damage to the earthfill dam proper occasioned by overtopping, a section of the embankment on the shore side of the spillway could wash out, and so relieve the load upon the main part of the dam. This safety factor would not come into play until after an 8-ft. crest had been reached upon the spillway, which was designed to accommodate a 5-ft. crest with the estimated 500-year storm.

The dam, impounding 150,000,000 gal. of water, is of the earthfill type, approximately 425 ft. long at the top and 90 ft. at the bottom, with a height of 75 ft. A reinforced-concrete corewall reaches from a point 3 ft. above the top of fill down to a solid anchorage in rock. This corewall extends horizontally from the dam side of the spillway, across the valley and is anchored into the opposite side of the valley up to a point well above possible high-water level. Between the shore side of the spillway and the actual slope of the natural valley the corewall is omitted, thereby providing an intentional point of weakness or safety plug as is illustrated by the accompanying figure. The rock elevation at this point is the same as that in the bottom of the spillway channel.

OMITTING A SECTION of the corewall between the spillway and shore side of Hornell Dam No. 3 provides an intentional point of weakness that will wash out before the dam is overtopped.



Spillway accommodates flow

The spillway weir is semicircular gravity type, with flanking tangents of the same section. The total length, including tangents, is 128.5 ft. The reservoir discharges over this weir into a spillway channel having a retaining wall extending from the end of the spillway weir, on the dam side, to a point well below the downstream toe of slope of the dam. The opposite, or shore side, of the spillway channel is an earth slope protected with one-man stone riprap.

At the end of the tangents above referred to, walls are constructed to retain the fill and serve as an approach to a bridge carrying a roadway alongside the reservoir and over the top of the dam.

Before a flood condition could overtop the corewall, which extends completely through the main part of the dam, there would be a flow 11 ft. deep over the spillway and 3 ft. deep over the roadway adjacent to the spillway on the shore side. During the recent flood the flow over the spillway weir reached a depth of approximately 4.25 ft., leaving some 3.75 ft. before this safety provision would come into play. Although the spillway channel below the bridge is designed for the 500-year storm (5-ft. spillway crest), the spillway will automatically accommodate itself to any flow beyond this quantity by tearing out of riprap on the shore side and thus widening itself; such widening is always in a direction away from the dam. This point was proved in the recent flood when a small section of the riprap, approximately 10 ft. in length, was taken out.

Design predictions confirmed

The crest of 4.25 ft. over the spillway that occurred on July 8 was equal to a discharge of 3,750 sec.-ft., equivalent to a runoff of 300 ft. per square mile of drainage area. At the time of maximum discharge the depths in the spillway channel closely approximated the design capacity equivalent to the flow over the spillway weir. An 8-ft. crest over this spillway would be equal to a discharge of 9,625 sec.-ft., or more than double that which occurred in the recent flood.

At the point where the spillway channel discharges into the valley the writer was informed that the flow was of such force that it jetted nearly across the valley. Nevertheless, no harm was done to the structure proper, and only minor repairs will be required to replace the small section of riprap in the spillway channel.

New Facts in Soil Mechanics from the Research Laboratories

Recent research develops new earth-pressure theory and discloses unexpected data on the consolidation characteristics of clays and the behavior of soils subjected to shearing stresses

By Arthur Casagrande
Graduate School of Engineering, Harvard University, Cambridge, Mass.

FROM A PAPER presented before the Society for the Promotion of Engineering Education, Atlanta, Ga., June, 1935.

DURING the the past two years a series of important discoveries in soil mechanics has begun. The most outstanding recent contribution is Terzaghi's theory of earth pressure, which offers an explanation for the many conflicting observations on the magnitude and especially on the distribution of earth pressure against retaining walls, timbering of trenches, excavation pits and tunnels. (Presented in a series of articles in *Engineering News-Record,* Feb. 1 and 22, March 8 and 29, April 19 and May 17, 1934. —Editor.)

By comparing the results of tests made at the Massachusetts Institute of Technology in 1929 with existing earth-pressure theories, Professor Terzaghi found that Coulomb's theory showed a good agreement so far as the magnitude of the pressure is concerned, but not in regard to its distribution. The latter he found to be controlled by the character of the yield of the wall. A tilting around its lower edge produced almost triangular distribution, in accordance with Coulomb's theory. However, parallel movement resulted in a different stress distribution, with the point of application of the resultant earth pressure considerably above the lower third.

New testing apparatus

The requirements of testing undisturbed clay samples have led to the design of a new type of consolidation apparatus, which has been successfully used during the past three years for general soil testing, as well as for extensive investigations of the consolidation characteristics of undisturbed fine-grained soils. A cross-section of the apparatus is shown in Fig. 2. This device is used in connection with the universal loading machine developed at the Harvard soil-mechanics laboratory, two views of which are shown by Fig. 1. This machine permits the simultaneous performance of consolidation, shearing, unconfined compression and permeability tests on the same framework, with a large saving in laboratory space, apparatus costs and, particularly, in working time.

Logarithmic scale graphs

In addition to improvements in apparatus and technique of testing for the consolidation characteristics of clay, the methods for evaluating the test results have been improved by plotting pressure and time in the compression curves and time curves on a logarithmic scale. This has many important advantages—e.g., when carrying out a settlement analysis it is of importance to determine as accurately as possible the maximum load to which a soil stratum was subjected during its geologic history. The break in the compression curve, corresponding to the pre-consolidation load, is often hardly visible on an arithmetic plot, as illustrated in Fig. 3A. This is particularly true when the sample has absorbed water during the process of removal.

This difficulty is easily overcome by plotting the pressure on a log scale, as shown in Fig. 3B. On such a semi-log plot, within our normal pressure range, the virgin compression curve of any fine-grained soil is practically a straight line, and any recompression curve deviates very distinctly from that line. By studying the interrelationship between recompression and virgin compression curves, it is possible to derive the natural preconsolidation load from tests on undisturbed samples.

Another difficulty which can be overcome by the use of semi-log plots resides in the evaluation of time-compression diagrams (Fig. 4). For the purpose of computing the coefficient of permeability, it is necessary to estimate

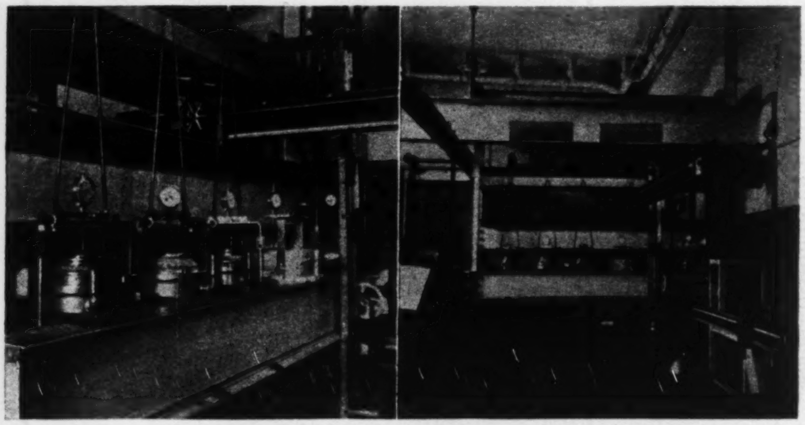


FIG. 1—UNIVERSAL SOIL-LOADING MACHINE for making four different tests simultaneously; general view (right) and detail of testing bench (left).

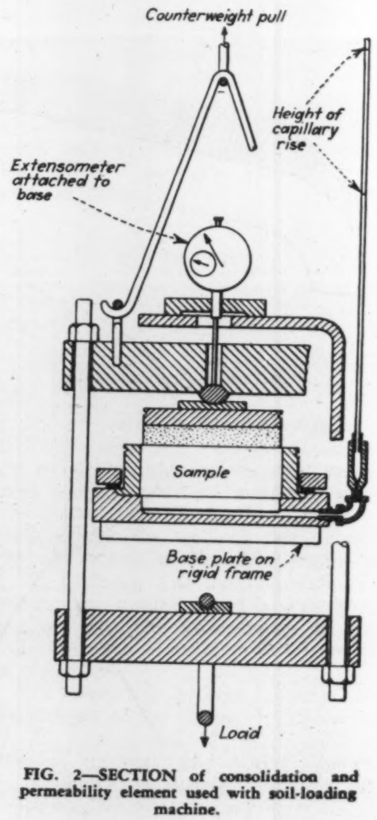


FIG. 2—SECTION of consolidation and permeability element used with soil-loading machine.

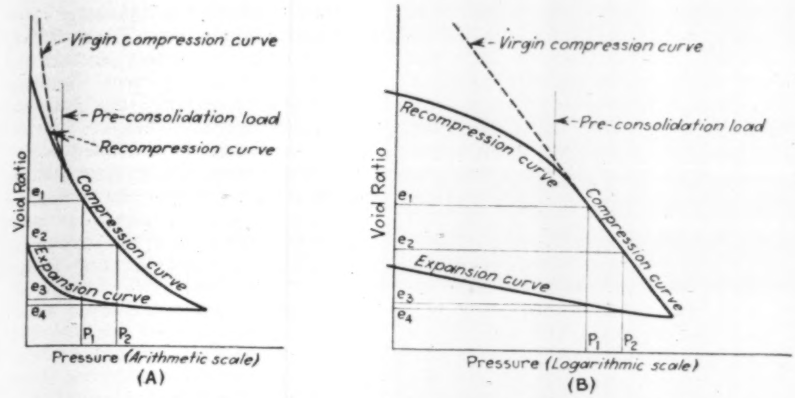


FIG. 3—COMPRESSION-TEST DIAGRAMS, showing comparison of curves plotted to arithmetic and logarithmic scales.

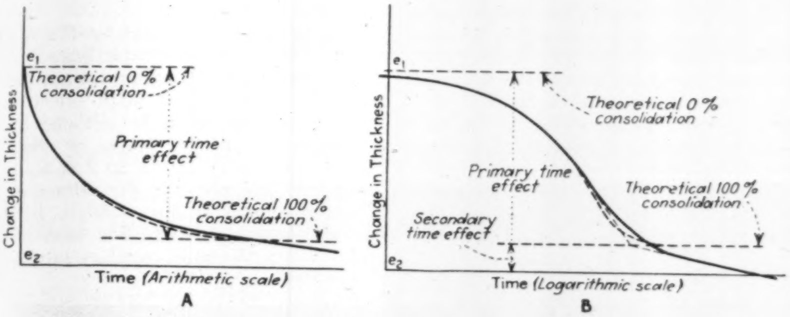


FIG. 4—TIME-COMPRESSION DIAGRAMS, showing comparison of curves plotted to arithmetic and logarithmic scales.

the position of the 100 per cent line for theoretical consolidation. In an arithmetic plot (Fig. 4A), this is very difficult and often impossible, while in the semi-log plot shown in Fig. 4B, one can easily differentiate between the theoretical consolidation of primary time effect and the secondary time effect, which are separated by a sharp curve.

In the theoretical time curve this sharp curve continues into an almost horizontal line approaching asymptotically the 100 per cent line, while the observed curves turn into a more or less straight line that is inclined. This secondary time effect can be so very pronounced that the primary effect is completely overshadowed. The physical cause of this secondary effect is probably the adjustment of internal frictional forces, although the phenomenon is still rather obscure. The magnitude of this secondary time effect is dependent on the character of the soil and particularly on the structure and the content of organic matter. It also depends on whether the compression is taking place along the virgin compression curve or on a recompression branch. In the first case the slope of the secondary time effect, on semi-log plot, is about three times as large as that on the recompression curve. This difference can be used as a check when investigating the pre-consolidation load of a material.

It has been found that temperature has a considerable effect on the consolidation characteristics of soil containing organic matter. The compression diagram of an undisturbed sample, first tested at 40 deg. F. up to a pressure of 1.42 kg. per. sq.cm., then rebounded to zero and recompressed at 74 deg. F., shows for example a large difference in the position of the virgin compression curves and consequently of the estimated pre-consolidation loads between these temperatures. In the case taken as example the pre-compression load was accurately known, and by interpolation it was found that a virgin compression curve for 50 deg. F. would correspond to the condition in the ground. This was an excellent check, because the average ground temperature of this material in its original state was estimated to be between 45 and 55 deg. F.

At present, investigations are being carried out in the cold room of the Harvard soil mechanics laboratory, to determine the temperature effect on inorganic clays.

Shearing resistance of soils

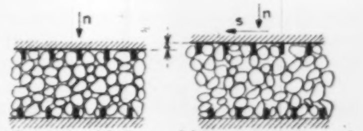
A very comprehensive program of research is in progress at the Harvard soil-mechanics laboratory on the effect of shear in soils. It is a continuation of investigations conducted at the Massachusetts Institute of Technology by Professor Terzaghi in 1929, by the writer from 1930 to 1932 and by Dr. Jürgenson from 1932 to 1934. At present, the following results seem fairly well established:

1. Practically all soils undergo appreciable volume changes during shearing under constant normal load. An accurate observation of these volume changes is of utmost importance for the purpose of proper interpretation of test results as well as for a correct application to problems in earth and foundation engineering.

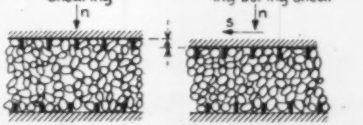
2. Loose sands and all very fine-grained soils (silts and clays) decrease in volume during shearing under constant normal load (Figs. 5c and d).

3. Dense sand expands during shearing under constant normal load (Figs. 6a and b).

4. All soils that are sufficiently coarse-grained so that the resistance in the pores to the flow of water does not cause a noticeable time lag in the volume change, possess definite angles of internal friction which are practically constant within the pressure ranges nor-



(a) Dense Sand Before Shearing (b) Dense Sand Expanding During Shear



(c) Loose Sand or Fine Grained Soil Before Shearing (d) Loose Sand or Fine Grained Soil Compressing During Shear

FIG. 5—EFFECT OF SHEAR on volume of loose and dense sand shown by grain rearrangement.

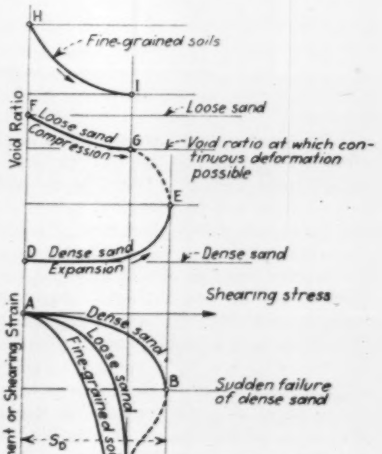


FIG. 6—DISPLACEMENT and volume change as functions of shearing stress of loose and dense sand.

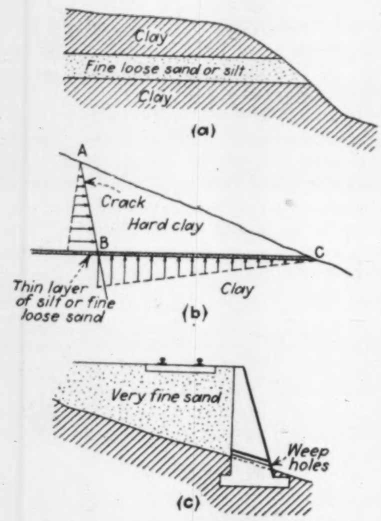


FIG. 7—SLIP AND SUBSIDENCE due to saturation and vibration of wet sand.

mally encountered in foundation engineering.

5. Very fine-grained soils, particularly clays, have for practical purposes no definite angles of internal friction. The shearing resistance of such materials is not only a function of the normal stress and the density, but also of the rate of shearing-load application and all those factors that determine the rate of consolidation.

Action of sand under shear

In Fig. 6 are plotted displacement and volume change as functions of the shearing stress. For a loose sand the shearing resistance increases gradually from point A to C, reaching its maximum value S_L, and then remains practically constant. Simultaneously the volume decreases from point F to G, reaching a void ratio at which continuous deformation is possible at constant resistance S_L. Shearing failure in loose sand is really a condition of plastic flow.

If the same sand is compacted into a dense state—that is, its void ratio decreased from point F to D in Fig. 6—and then is subjected to shearing, one finds that the horizontal displacement at equal stress is smaller than for the loose state. The volume remains almost constant for stresses up to one-half the shearing strength, and then the sample begins to expand at an increasing rate, the expansion curve having a vertical tangent when failure occurs. Both the stress-displacement and the expansion curve are not as smooth as shown in Fig. 6 but possess numerous sharp corners that are due to local slips and rearrangements of the structure. The shearing strength of dense sand is about 30 per cent larger than of the same sand in loose state. Failure of dense sand occurs suddenly and has more the character of rupture, if compared with the character of failure of the loose sand. If the test is conducted in a machine in which the strain can be kept constant (instead of the stress), then the test can be continued beyond failure. The stress decreases with increasing displacement (curve B-C), and the volume increases (curve E-G) until again that critical void ratio is reached at which continuous movement is possible.

Practical significance of shear

The volume decrease of loose sands and fine-grained soils associated with deformation has a fundamental influence on their stability if their voids are saturated. If we consider, e.g., a porous stratum of loose saturated silt or fine sand between beds of clay, as shown in Fig. 7a, and analyze what should happen if this stratum is subjected to considerable shearing forces, we find that the tendency of the material to decrease its volume is counteracted by the presence of water in the voids. If the volume decrease cannot take place because the excess water in the voids cannot escape sufficiently fast, the entire overburden will temporarily be carried by the water within the sand or silt stratum. The supersaturated sand or silt has practically no shearing resistance and will flow out. The overlying clay may either be carried out with the flowing mass or may drop vertically down immediately after the flow has occurred, or at a later period.

Another example is illustrated in Fig. 7b, a thin, fine-grained, porous layer embedded in a mass of hard fissured clay. During a rainy period or when the ground thaws out in the spring, the cracks fill with water, creating hydrostatic pressures not only in the crack but also in the thin porous layer. While such a condition in itself is rather dangerous, the slightest movement within the porous layer produced by the shearing forces will result in a condition of supersaturation, so that the mass of clay between points A, B and C will slide out as if suddenly placed on roller bearings.

In both cases, illustrated by Fig. 7a and b, slides are greatly facilitated, if the porous stratum is sloping toward the bank, a condition that often prevails. Such slides may often be initiated by vibrations due to earthquakes or other causes. An example showing how train vibrations may result in the failure of a retaining wall is illustrated in Fig. 7c. The backfill of very fine sand remains in a condition of capillary satura-

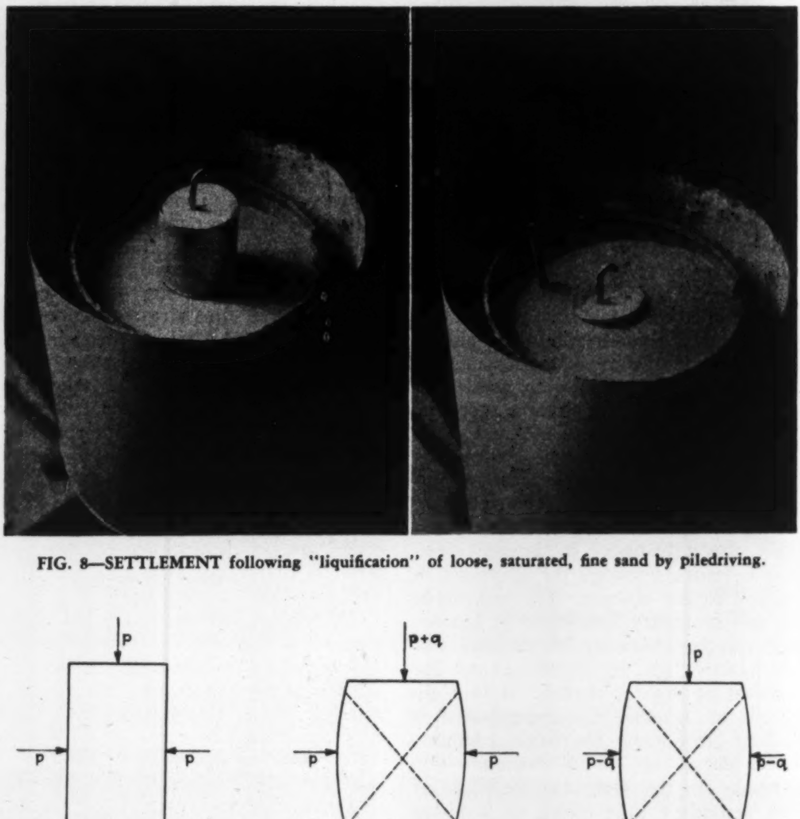
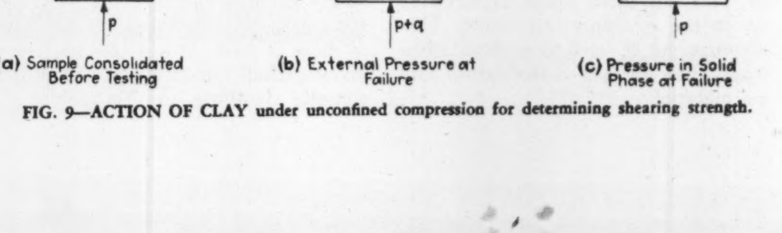


FIG. 8—SETTLEMENT following "liquification" of loose, saturated, fine sand by piledriving.



(a) Sample Consolidated Before Testing

(b) External Pressure at Failure

(c) Pressure in Solid Phase at Failure

FIG. 9—ACTION OF CLAY under unconfined compression for determining shearing strength.

tion in spite of perfect drainage provisions through the wall. The sudden deformations within the mass of sand produced by vibrations will cause the mass to liquefy, resulting in a tenfold increase of the pressure against the wall, for which it is not designed. In a similar manner, embankments of fine sand have flowed out under the impact of traffic, dikes against the sea under the impact of waves. Professor Terzaghi has described examples and the mechanics of slides due to subsidence within a sand stratum, in *Engineering News-Record,* July 16, 1931.

Most foundation engineers are familiar with the danger of driving piles in loose saturated deposits of very fine sand or silt. Such a procedure may lead to serious settlements of neighboring structures. A convincing and simple demonstration for this and the above conditions is shown in Figs. 8a and b. A fine, very uniform, quartz sand is deposited under water in a loose state, and a weight is placed on the surface (Fig. 8a). Then a stick of wood is thrust into the sand, imitating the driving of a pile, and suddenly the whole mass liquefies, causing the weight to sink (Fig. 8b).

The process of shearing in clays is very complicated, and it must be admitted that our knowledge of this question is very limited. Therefore, the following remarks and hypotheses shall only serve to convey a conception of the nature of the problem and its difficulties.

If a clay sample is subjected to very slowly increasing shearing forces under constant normal load, a large decrease of volume takes place; but within the normal range of displacement of a shearing test a stationary condition cannot be reached. The volume decrease will be only a part of the decrease from H to I (Fig. 6), and correspondingly the shearing stress will not reach its maximum value. In order to arrive at the maximum, shearing would have to be continued until the sample is completely remolded, requiring a displacement many times the thickness of the sample.

Similar difficulties arise if unconfined compression tests are employed as a means for determining the shearing strength of clays. In Fig. 9 is shown a cylindrical sample of clay which we assume to be consolidated under a pressure p acting equally in all directions. Then the vertical pressure is raised quickly until failure occurs at a vertical pressure $p + q$, as indicated in Fig. 9b. Since no time is allowed for consolidation, the additional pressure q will be carried by the water, which means that the horizontal pressure p will no longer be transferred from grain to grain, but only the difference $(p - q)$, q being carried by the water. In other words, the application of an additional vertical pressure has theoretically reduced the horizontal pressure between the soil grains by an equal amount, as shown in Fig. 9.

Record Span of 528 Ft. for Concrete Arch of Through Type

La Roche-Guyon bridge over Seine River in France consists of two hingeless arch ribs reinforced with tiers of spiral coils—Deck is suspended from reinforced-concrete hangers

THE NEW highway bridge over the Seine, at La Roche-Guyon in France, resembling in general appearance the Castelmoron reinforced-concrete through-arch bridge (*ENR,* April 25, 1935, p. 577), is discussed by one of its builders, S. Boussiron, in *Le Génie Civil* of Feb. 9, 1935.

The reinforced-concrete through arch of this bridge has a span 528 ft. long, which makes it the longest of its kind in the world. Its rise is 75.5 ft.—i.e., ⅐ of the span. The total length of the bridge is 662 ft. The roadway is 18.4 ft. wide throughout; the total width of the bridge, including two sidewalks, is generally 32.8 ft., except at the intersection with the arch ribs, 4.26 ft. above their springings, where it is only 26.25 ft.

The arch ribs are fixed and have a hollow cross-section varying in depth from 4.7 ft. at the springing to 8.7 ft. at the crown, and in width from 9.8 ft. at the springing to 4.6 ft. at the crown. At the sections of minimum strength, at

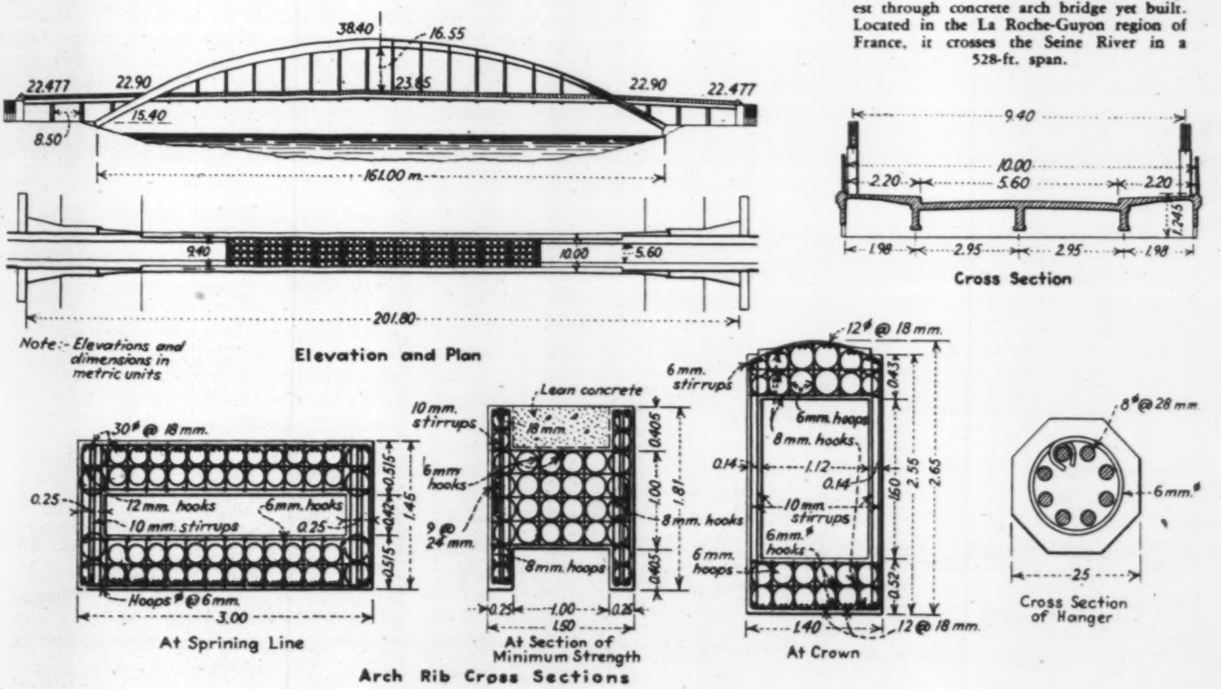

38.40
16.55
22.477
22.90
23.85
22.90
22.477
15.40
8.50
161.00 m.
9.40
10.00
5.60
201.80
Note:- Elevations and dimensions in metric units
Elevation and Plan
9.40
10.00
2.20
5.60
2.20
1.245
1.98
2.95
2.95
1.98
Cross Section
30# @ 18 mm.
0.25
12 mm. hooks
6 mm. hooks
10 mm. stirrups
0.25
Hoops # @ 6 mm.
3.00
0.515
0.42
0.515
1.45
At Sprining Line
Lean concrete
10 mm. stirrups
18 mm.
6 mm hooks
9 @ 24 mm.
8 mm. hoops
0.405
1.00
0.405
1.81
0.25
1.00
0.25
1.50
8 mm. hooks
At Section of Minimum Strength
12# @ 18 mm.
6 mm. stirrups
6 mm. hoops
8 mm. hooks
0.14
1.12
0.14
10 mm. stirrups
6 mm.# hooks
6 mm. hoops
0.43
1.60
0.52
2.55
2.65
1.40
12 @ 18 mm.
At Crown
Arch Rib Cross Sections
8# @ 28 mm.
6 mm.#
25
Cross Section of Hanger

FIG. 1—STRUCTURAL DETAILS of largest through concrete arch bridge yet built. Located in the La Roche-Guyon region of France, it crosses the Seine River in a 528-ft. span.

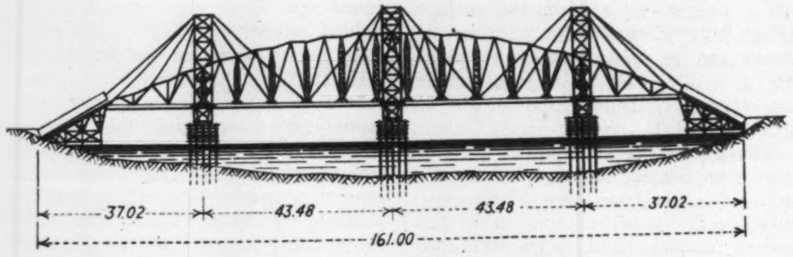


FIG. 2—ERECTION of the La Roche-Guyon bridge was accomplished by a form system suspended by cables from three falsework towers founded on piles in the river bottom.

a horizontal distance of 220 ft. from the crown, the upper and lower slabs of the arch section come together and the rib becomes solid. These sections of minimum strength were provided for the relief of the fixed end moments. In general, the moments of inertia of the arches were adjusted, to result in practically uniform maximum stresses throughout and in maximum flexibility.

The arch ribs are reinforced, principally with tiers of spiral coils of mild steel, about 8 to 10 in. in diameter, with auxiliary stirrups and hooks, amounting to 1⅓ per cent of the concrete in the section, which averages nearly 24 sq.ft. in area. Longitudinal round bars, provided to the extent of ½ per cent of the area of the concrete, serve merely as spacers. The concrete, thus reinforced, is good for design stresses up to about 1,775 lb. per sq.in.

The suspended portion of the bridge deck (440 ft. long and 6 in. thick), which is structurally independent of the rest of the deck, is carried by 32 vertical concrete hangers of octagonal cross-section, reinforced as shown in Fig 1.

The sway bracing consists of a light open latticework, and extends about 145 ft. either side of the crown; it is supported principally on three longitudinal arch ribs and two heavy transverse beams. Ten additional cross-pieces above the latticework tie this structure to the main arches.

The method of construction is illustrated on Fig. 2. The formwork was carried by cables strung from three wooden towers, which were spaced 143 ft. apart and 121 ft. from each shore. The deck was built first, from shore to shore, and on top of this prestressed forms for the arches were erected. The double-plank deck of the arch forms and the bridge deck were tied together with cables, to make a rigid truss. Concreting proceeded from springing to crown in this order: (1) lower slab of arch rib, (2) arch rib sides, (3) top slab of arch. During construction, hydraulic jacks were used extensively for prestressing and for the control of thrusts and moments in the arch ribs, essentially in accordance with the Freyssinet system.

The concrete reinforcement was vibrated with pneumatic hammers, and the forms with special vibrators. A total of 1,620 cu.yd. of concrete was used, varying in richness from 590 lb. of cement per cu.yd. in the deck, to 670 lb. per cu.yd. in the arch ribs, hangers and windbracing.

Design considerations

Further information on the design of this bridge was presented by N. Esquillan in *Le Génie Civil* of Feb. 16, 1935.

The low ratio of rise to span, 1:7, was selected for esthetic reasons entirely, although it causes large thrusts on the abutments. The cross-section of the arches, which increases in depth and in moment of inertia from abutment toward the crown, was studied in comparison with sections of two other equivalent arches: a parabolic arch of uniform cross-section, designated I; and an arch of variable cross-section designated III, which increased in depth and in moment of inertia from the crown toward the abutments in accordance with a formula proposed by Chalos. The relations of the three types of arch sections are shown in Fig. 3. In this figure and in the following ones the La Roche-Guyon arch is designated as II.

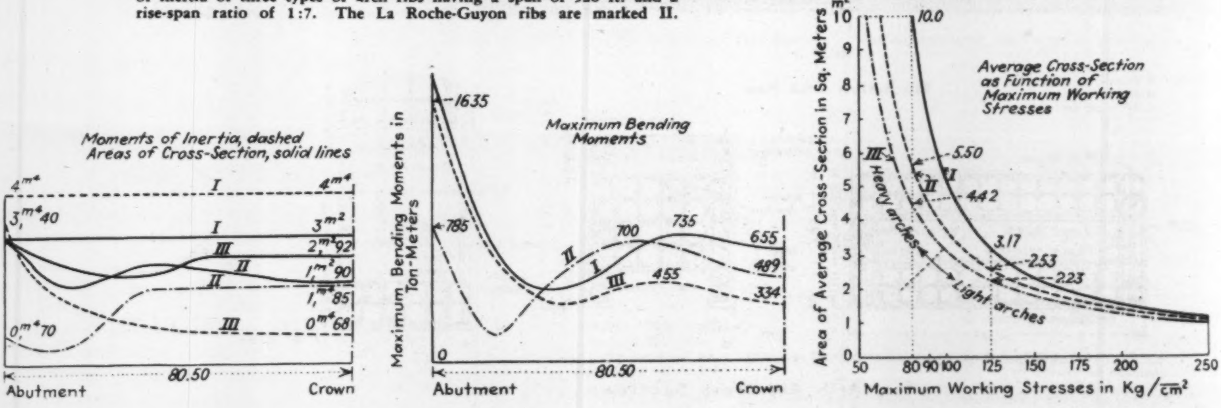
The La Roche-Guyon arch (II), as shown on the diagram of Fig. 3, has the advantage of a lower range and of a lower maximum bending moment: 5,500,000 ft.-lb. against about 10,750,000 ft.-lb. for arch type III and 11,700,000 ft.-lb. for type I.

Fig. 3 represents the variation of the area of the average cross-section required as a function of the assigned maximum working stress, for all three arches. Again the La Roche-Guyon arch is the most advantageous. From this figure it also appears that if a more normal maximum working stress of only 1,138 lb. per sq.in. had been allowed instead of the 1,778 lb. per sq.in. actually adopted, the average area of cross-section would have to be doubled, or about 47.5 sq.ft. instead of the actual 24 sq.ft.

Tests of bridge

The bridge was tested two times for dead loading and five times for live loading. The weight of the falsework happened to be nearly equal to the dead loading specified, so that in the first test the entire falsework was made to hang on the arches and in the second test one-half of the falsework was removed and the other half left suspended. All deflections were found to be consistently proportional to the ones predicted by theory. The instantaneous coefficient of elasticity proved to be about 5,600,000 lb. per sq.in. Maximum stresses at springings were: 412 lb. per sq.in. compression and 384 lb. per sq.in. tension. Since temperature stresses amount to 142 lb. per sq.in., it is felt that normally the maximum bending stresses will not exceed 555 lb. per sq.in. at the springings.

FIG. 3—COMPARISON of bending moments, areas and moments of inertia of three types of arch ribs having a span of 528 ft. and a rise-span ratio of 1:7. The La Roche-Guyon ribs are marked II.



Engineering in Foreign Countries

Accounts taken from the foreign-language periodicals of important activities in engineering research, design and construction being carried on beyond the United States

TO GIVE the readers of *Engineering News-Record* quick glimpses of engineering activities beyond the United States, these short abstracts of articles and papers published in languages other than English are presented monthly. In these abstracts no attempt is made at a compendium of foreign-language articles, but selection is made from articles that emphasize some procedure or invention in engineering practice that has general interest or usefulness.

Previous groups of these abstracts from foreign periodicals have been published in *Engineering News-Record* of Jan. 31, Feb. 21, March 28, April 25, May 30, July 4 and July 25, 1935.

Improving hill terraces by blasting

Doctor A. Piédallu describes in *Journal d'Agriculture Pratique*, Feb. 9, 1935 (see also *Le Génie Civil,* July 6, 1935), a method developed in Algeria for combatting erosion and improving growing conditions on hillside terraces.

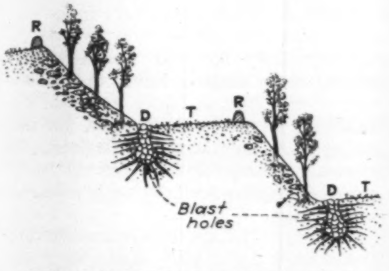


FIG. 1—METHOD of terracing in Algeria for reducing erosion.

At intervals of 15 to 50 ft. low ridges *R* (Fig. 1) are thrown up along the downhill edge of the terrace, and the surface of the terrace *I* is sloped down gently toward its uphill edge. One or two cartridges of so-called agricultural powder. costing 1.50 to 2.50 francs (about 10 to 17c.), are set at a depth of 28 to 40 in., at intervals of 15 to 50 ft. The explosion of these cartridges produces a ditch of irregular section with rather long fissures radiating from it into the surrounding soil. The ditch is backfilled with rock, at least 4 or 5 in. in size.

The ridges *R* check the violent runoff from the hillside, and the rockfilled ditches *D* with their fissures promote its complete absorption and distribution within the soil. Hillsides thus treated have grown rich crops of fruits and vegetables in the arid climate of Algeria without irrigation. Where the top soil is of a calcareous nature, the ridges *R* may be dispensed with. Where the soil is rocky, trees should be planted in the blasted loosened soil of the ditch *D*.

Soviet prefabricated buildings

An unprecedented volume of construction at an unusual speed coupled with a shortage of labor have greatly stimulated the development of prefabricated concrete building in U.S.S.R., particularly in Moscow and in Leningrad. Prefabrication has also been introduced in power plant and hydraulic construction, and nowadays Soviet engineers when projecting construction have to show cause why prefabrication methods should not be used if they are not.

The Soviet system, unlike the French small-unit system of prefabrication used in the building of apartment houses at Drancy (*ENR,* June 27, 1935, p. 903), employs large units and sets of units, ranging from 1 to 3 tons in weight, which are placed by powerful tower and portal cranes.

A. I. Kucherov and N. N. Porphyryev, writing in *Socialistichesy Gorod,* February, 1935, report the construction of a Moscow building of 82 apartments, five to six stories high, by the large-block prefabrication method. Precast wall blocks of slag concrete, 10 to 15 in. thick, averaged about 2,750 lb. in weight. Monolithic units of balcony slabs with adjacent beams weighed 5,500 to 6,600 lb. apiece. Monolithic flights of stairs ranged from 1,300 to 1,800 lb. in weight. Lighter members, such as roof trusses and joists, were assembled on the ground and were placed as one compound unit. Erection was carried out with standard tower cranes.

The blocks were cast in wooden chambers, within reach of the tower crane, accommodating about 6 cu.yd. of castings—approximately one day's output. Steel forms are used. After casting, the chambers were closed and filled with steam at a temperature of 120 to 175 deg. F. The steam-curing period ranged from 60 to 120 hours, after which the blocks usually tested 1,400 to 1,600 lb. per sq.in.

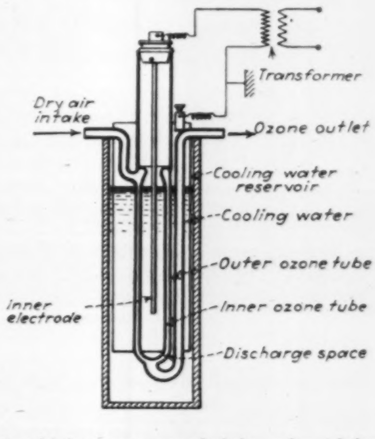
Wall construction proceeded at the average rate of 900 sq.ft. per shift. The crew consisted of seven men and one woman. On the basis of ordinary brick construction, as practiced in Moscow, there was a saving of about 75 per cent in labor and more than 50 per cent in cost of construction.

N. Ukhanov, writing in the same issue of the magazine, outlines the Leningrad modification of this system of construction. In Leningrad they have a central casting yard containing eighteen steam-curing chambers having a total volume of 7,280 cu.ft. The steam temperature is 167 deg. F., and the curing period is 18 hours for plain concrete and 24 hours for reinforced-concrete members. Steel forms and electric tampers are used. The castings are delivered to the erecting cranes in small trucks.

Water purification with ozone

In western Europe, particularly in France, ozone treatment of water supplies has lately come to the fore as a most vigorous competitor to chlorination. Sir Alexander Houston's tests at the Barn Elms filtration plant have convinced him that many times ozonization should be recommended in spite of its cost, which may be 300 per cent higher than plain chlorination. Indeed, at the Toulon waterworks in southern France, chlorination was given up in favor of ozonization and a new 11-m.g.d. ozone water-purification plant is in course of construction. At Nancy too, ozone supplanted chlorine, and a 22-m.g.d. ozonization plant has been recently put in operation. In Paris, the old 24-m.g.d. ozonization plant, established in 1913, is being enlarged to a capacity of 106 m.g.d.—which will make it the largest plant of its kind in the world.

FIG. 2—DETAILS of ozone-generating tube element for water purification.



The German-made ozone treatment apparatus, forming part of the equipment of the old and the new plants of the Paris waterworks, is described by K. Bergmann and K. Holzwarth in *Siemens Zeitschrift,* December, 1934. Fig. 2

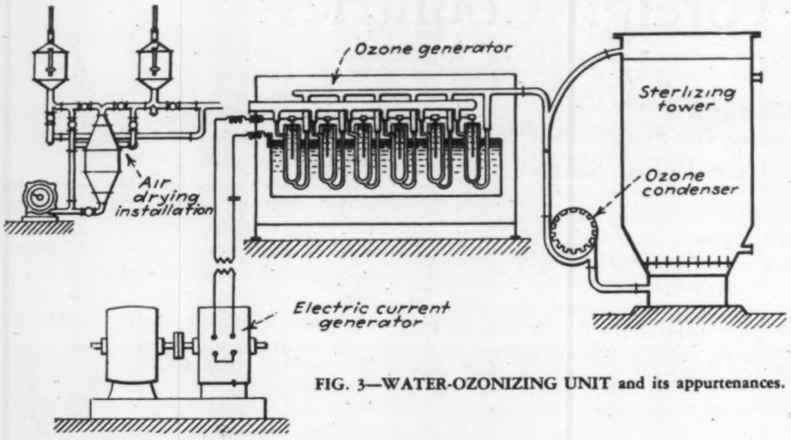


FIG. 3—WATER-OZONIZING UNIT and its appurtenances.

shows the details of a single ozone-generating tube element of Siemens design. Fig. 3 shows an ozone-generating battery consisting of six tube elements together with its appurtenances. One such battery will generate 600 grams of ozone per hour. Since 4 to 6 grams of ozone are required to sterilize 1,000 gal. of water, one battery will take care of 100,000 to 150,000 gal. of water hourly. This ozone generator requires single-phase alternating current of a frequency of 10 kilocycles. The high-frequency current is produced by a special high-frequency generator and a resonance transformer. On the basis of Sir Alexander Houston's test of 1931, it may be estimated that the ozonization of 1,000 gal. of water will consume from 85 to 130 watts. Higher efficiencies are probably attainable at present.

In addition to the seventy-odd ozone water-purification plants known to exist in France, there is also one at Braila, Roumania, of 2.6 m.g.d. and one at Yawata, Japan, of 6.8 m.g.d., besides a number of smaller ones located in other countries.

Effect of forests on yield of watersheds

A. F. Meyer's survey of German experience as to the effect of forests on watershed yields, published in *Gas und Wasserfach,* April 13, 20, 27 and May 4, 1935, is particularly timely now when the probable effects of the shelterbelt project and other afforestation schemes for the United States are being discussed and appraised.

On the basis of his original analysis of precipitation and runoff records of various German watersheds, supplemented with an examination by leading German authorities on climatology, forestry and hydrology published within the past 30 years, Meyer has arrived at the following conclusions:

1. Increasing the forest area of a watershed has no quantitatively determinable effect on its precipitation and runoff conditions.
2. Retention of rainfall water in a forested area affects the supply of its springs and brooks only for the duration of one month or so.
3. A forested area consumes irretrievably 7 to 8 in. of the annual rainfall for its seasonal vegetating processes.
4. There are no data in support of the allegation that forested areas enjoy heavier rainfall by attracting thunderstorms.

Rhine River fishways

Now that it has been decided to spend $3,200,000 for fishways at the Bonneville Dam (*ENR,* July 4, 1935) it may be interesting to glance at the much cheaper arrangements provided for salmon runs at the Kembs Dam of the international navigation and hydro-electric power development on the Upper Rhine.

According to H. Blattner, writing in *Schweizerische Bauzeitung,* Jan. 5, 1935, the fish ladder and fish elevator for the Kembs Dam, which is about 50 ft. high, cost 200,000 German Reichsmarks in all, or not quite $81,000.

As shown on Fig. 4, the elevator and the ladder are both located at one abutment of the dam. The ladder is Z-shaped in plan and consists of two parallel series of basins 6.2 by 6.2 ft., 6x6 ft. deep, differing in elevation by 6.7 in. It is shown in Fig. 4. A fish entering from tailwater at the lower end of the ladder will ascend through the right-hand series of basins in a general downstream direction up to the mid-height of the structure, where its course will be deflected into the left-hand series of basins and will be reversed to a general upstream direction and over the crest of the dam into headwater. The transverse baffles of the basins have a square notch at the top and a square hole near the bottom, both off center. Both notches and holes are not in line but staggered, thus favoring the formation of still-water nooks where the weary fish may rest. The basins are so designed that the velocity of flow will not exceed 5.9 ft. per second, no matter what the water level above the dam may be. The effectiveness of the design was tested on a laboratory model. The fish ladder cost nearly $65,000.

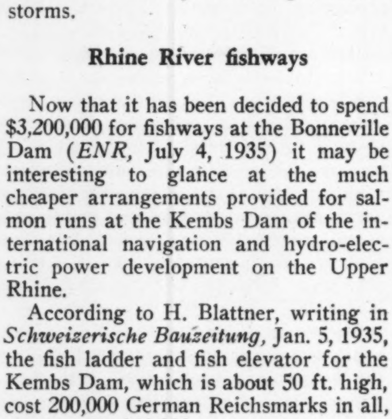


FIG. 4—FISH LADDER and elevator at the Kembs Dam on the upper Rhine. The ladder is a Z-shaped channel shown in plan on the left abutment.

The fish elevator, which is a $16,000 patented installation, is located just downstream of the fish ladder, in a square concrete shaft, 8.2 by 8.2 ft., 33 ft. deep, having a flaring inlet, 1.6-ft. minimum width, at one of its corners. A wire net basket, fastened to a guided frame and geared to a clockwork mechanism, runs automatically at set intervals up and down this shaft. Upon passing the top of the shaft, the basket is automatically shunted sidewards and is dumped into a flume leading to headwater. Thus, the fish is lifted across the dam "in the dry," which is not harmful since the elevator trip is very short. Various means are used to entice the fish into the elevator shaft, among them electric lights at the bottom of the shaft and jets of water, discharging from the shaft walls, for agitating the water at the shaft inlet. To attract salmon, the inlet has to be set more or less tangentially with reference to the river.

In May, 1934, the ladder and elevator were used by 60,000 fish, the overwhelming majority showing preference for the fish ladder. But the year before the elevator was all the rage and basket loads of 150 lb. of salmon and other fishes were often observed. On the Rhine too, fishery interests were loud in clamoring, through the daily press, for prohibitively costly and even impossible schemes to safeguard the fish runs.

Life of concrete seaport structures

The results of a recent survey of the condition of old concrete structures in seaports of the Soviet Union are discussed by Professor A. T. Fyodorov in *CNIVT Sbornik Nauchno-Issledovatelskikh Materiallov* (Transactions of Central Research Institute of Water Transport) of 1934. The survey included the breakwaters, docks and other maritime structures of the ports of Vladivostok, on the Pacific Coast, Tuapse, Batum, Poti, Novorossiysk, Yalta, Odessa, Kerch, Taganrog, Rostov and Mariupol, on the Black and Azov Seas, and of Leningrad on the Baltic Sea. Most of the structures examined, which are 30 to 65 years old, were built of concretes containing about 315 lb. of Russian portland cement per cubic yard. The Russian cement fully satisfies usual European specifications, having a silica modulus of 4.8 and containing less than 1.3 per cent of magnesia and less than 0.8 per cent of aluminum.

Nearly cubical samples, from about 6 to 12 in. to a side, taken from some of the structures, were tested in compression and gave results presented in the accompanying table.

COMPRESSIVE-STRENGTH TESTS

Location	Age, Years	Concrete Mix	Compressive Strength, Lb. Per Sq.In.
Batum	30	1:2½:5½	2,620
"	"	"	3,240
"	"	"	1,710
"	"	"	2,810
Tuapse	33	1:2½:5	3,160
"	"	"	2,000
Poti	59	1:2½:3½	1,890
"	"	"	1,730
"	"	"	1,860
"	"	"	1,490

In general, all existing concrete construction was found in good condition. In Tuapse and in Batum concrete blocks 30 years old were salvaged for use in new structures. Thus it appears that the opinion of authoritative cement chemists that concrete in seawater is good for only 20 to 30 years is not confirmed by facts. On the contrary, it may be claimed that concrete maritime structures will last 60 years or more.

It is further concluded that the richness of concrete for maritime structures should be increased to range from about 350 to 400 lb. per cubic yard; that 10 to 15 per cent of hydraulic admixtures is desirable; that the portland cement used should have a silica modulus of not less than 4.5; and that in cold climates the above-water parts of concrete port structures should be faced with thin-jointed ashlar. It has also been found that in seawater reinforced-concrete piles, not especially protected against collision shocks, will become cracked and will deteriorate in three to seventeen years.

Safety Methods Reduce Accidents at Fort Peck Dam

Accident frequency has been greatly reduced on the Fort Peck Dam operation by intensive safety campaign conducted by a safety section set up early in organization of construction operations

FROM December, 1933, to June, 1935, inclusive, a total of 5,717.011 man-hours were worked on the day-labor divisions of the Fort Peck project, and during that time there occurred 220 accidents of a minor nature where the employees received first-aid treatment and resumed their usual duties without loss of time, 144 injuries involving loss of time and 5 fatalities. The frequency rate (number of lost-time injuries, including fatalities per million man-hours worked) for this period was 25.19, which is quite low for this type of project.

Starting with July, 1934, the various army engineer departments, working more than 250,000 man-hours per month, were placed in competition with each other, for a six-months period ending Dec. 31, 1934. The Fort Peck District received the safety award given by the Office of the Chief of Engineers, U. S. Army, having had the best record for the six-months period, which was 16.21. The scores for the other competing districts in the U. S. Engineer Departments were: Vicksburg, Miss., 26.02; St. Louis, Mo., 31.88; Kansas City, Mo., 33.03; Memphis, Tenn., 36.93; and 2nd New Orleans, La., 49.53. During this time the Fort Peck District had 59 non-lost-time injuries, 30 lost-time injuries, and no fatalities, having worked 1,877,621 man-hours.

During the same length of time (July 1 to Dec. 31, 1934) the contractors who are prosecuting various phases of the work on the Fort Peck project worked 3,094,621 man-hours, had 235 lost-time injuries, seven fatalities, and had an average frequency rate of 48.67.

For the six-months period ending June 30, 1935, government employees worked 2,829,297 man-hours, had 57 lost-time injuries, three fatalities, and maintained an average frequency rate of 21.21. The contractors during the same period worked 2,493,575 man-hours, had 223 lost-time accidents and one fatality, with a resulting frequency rate of 89.83.

The combined government and contractors' forces worked 5,322,872 man-hours, had 280 lost-time accidents, four fatalities and an average frequency rate of 53.35 for the first six months of 1935.

In addition to the safety section established by the government, which sees that safety rules as established by the federal and state governments are adhered to by the contractors, the Mason & Walsh Construction Co., contractor on the diversion tunnels, has its own first-aid station with attendants who are on duty at all times. Spillway Builders, Inc., and the Martin Wunderlich Co., contractors on the spillway, have established first-aid stations, the former having a man whose entire time is devoted to accident prevention; and the Addison-Miller and Fielding & Shepley Co., Inc., contractor for the spillway gate structure, also has a modern well-equipped first-aid station with attendants. In addition, the government maintains a hosptial, fully equipped, with three medical officers and a hospital staff for the use of government employees.

Most of Fort Peck Labor Comes from Montana

AS THE FORT PECK project is being built under PWA labor regulations, most of the labor required, both skilled and common, is secured from local sources through the national re-employment service at Glasgow. The entire state of Montana has been designated as the employment district. At the middle of July, 6,584 persons were employed on the project, of whom 5,926 were residents of the state. The maximum employment last year was 7,236; this year, 7,035. Since June, 1934, direct employment has not fallen below 5,000.

All of the common labor is local, many of the workmen coming from cattle and sheep ranches. The government gets some of its skilled labor from the civil service lists. The supervisory force on the day-labor operations, except for army officers, is made up mostly of well-known construction men, many of them former contractors.

The contractors voice very little complaint about the quality of labor, both skilled and common, supplied by the national re-employment service. The chief complaint is the length of time required to obtain certain skilled men, for after a request is made the men selected often have to come from far corners of the state. Certain classes of skilled labor are scarce, and the contractors are allowed to bring them in from outside. These include welders and heavy-machine operators.

Most of the tunnel workmen employed on the project come from the mining districts of the state.

Unmapped Eroded Limestone Complicates Corewall Job

Usual borings fail to disclose either surface profile or cavernous body of limestone ledge underlying dike for Ontelaunee Dam at Reading, Pa.

By Farley Gannett
Gannett, Eastman & Fleming, Inc., Harrisburg, Pa.

BECAUSE OF numerous inadequacies of the old water-supply system, notably the inability of the slow-sand filters to handle the occasional high turbidities, Reading, Pa., is engaged in the construction of an entirely new supply works. The new system includes an earth dam and concrete spillway on Maiden Creek, a concrete conduit for raw water and a purification plant. The old slow-sand filters are to be remodeled to serve as a filtered-water reservoir. Storage on the distribution system is also to be improved. A general description of these operations was published in *Engineering News-Record*, Sept. 18, 1934, p. 323. The present article describes one of the unusual tasks of construction. —EDITOR.

THE CITY of Reading, Pa., embarked, in 1934, on an extensive program of waterworks improvement. Most of the contracts have been let, and five have been completed. One of the completed contracts is described in this article, and the reason for so doing is because of the extraordinary subsurface conditions that were found to exist. This contract consisted in the construction of a low, earth dike about 2,700 ft. long, with a maximum height of about 20 ft., containing 65,986 cu.yd. of rolled-clay embankment. This dike extends eastward from a concrete spillway section, and both it and the dam are underlaid with limestone.

The unit-price bid upon which the contract was awarded figured out to a total, on the basis of the quantities in the proposal, of $82,992. The total value of the contract as per the final estimate was $209,018, or about two and one-half times the original bid. This increase was due almost entirely to the increase in four items, all of which resulted from the fact that the area was underlaid with limestone. These four items were rock excavation, concrete drilling and grouting. The other major quantities, earth excavation, backfill, embankment and riprap, came out exceedingly close to the original estimates.

Before the job was let, the engineering staff of the Reading water bureau made quite comprehensive exploration of underground conditions by borings, many of them being on or near the center line of the dike as built. Others were drilled off the center line but not far from it. Under reasonable conditions these holes might fairly well have been expected to indicate the elevation of rock, but not so with the limestone rock in this particular place, because when the earth was removed from the rock, in excavating the wide cutoff trench (Fig. 1), the surface of the rock was found to be in a most unexpectedly irregular profile, and many of the drill holes had apparently touched only the high spots.

At one point where rock was expected to be found at 16-ft. depth, excavation was carried down to 30 ft. without finding rock. The surface of this rock looked like the "bad lands" of South Dakota or the Garden of the Gods in Colorado. Occasionally there were fairly level stretches, then would come a pocket 10 or 15 ft. deep in the middle of which would stand out a limestone monument several feet high. The whole surface of the rock was worn smooth by an ancient water course, and these irregularities in the profile resulted from the same cause. The view shown in Fig. 2 tells better

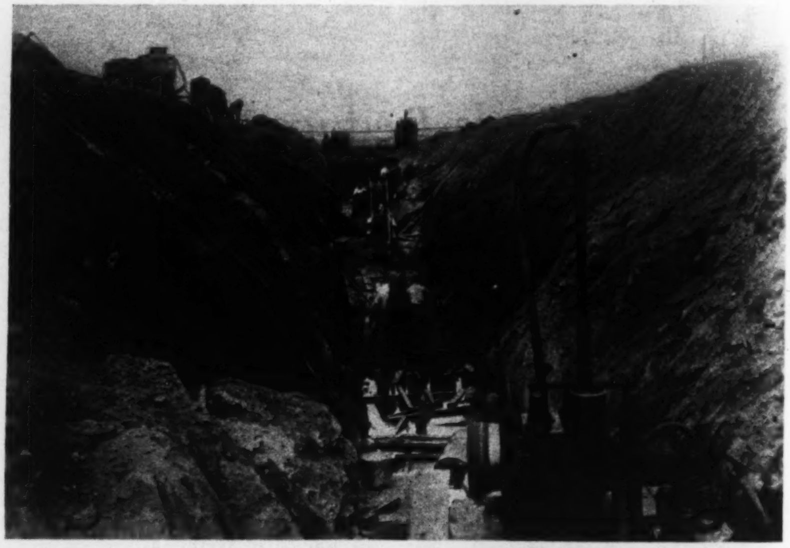


FIG. 1—COREWALL UNDER CONSTRUCTION for earth dike that flanks the concrete dam for the improved water supply of Reading, Pa.

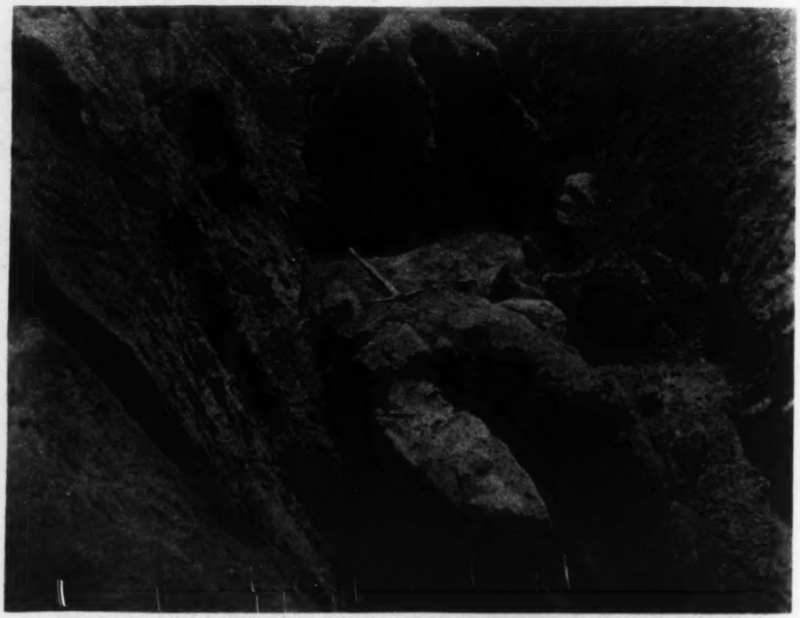


FIG. 2—TYPE OF ROCK SURFACE disclosed by excavation for the corewall trench of the dike for the Reading waterworks.

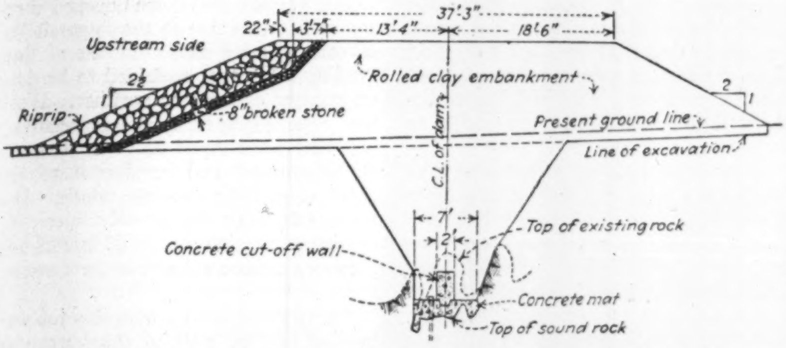


FIG. 3—SECTION OF DIKE AND COREWALL in the Ontelaunee Dam for the water supply of Reading, Pa.

than words the conditions that were found.

The result of all this was that whereas the original estimate had, on quite a reasonable investigation, indicated about 450 cu.yd. of rock to be removed, the final estimate showed 4,719 cu.yd. actually taken out.

The bulk of the material overlying the rock was taken out by power shovel and clamshell, but to clean the rock surface preparatory to putting a concrete mat thereon, deep pot-holes had to be cleaned out by hand; and when we say "by hand" we mean "by hand" because some of the pot-holes were too small even for a shovel. The rotten or cracked rock from the surface was removed, and the attempt was made to level this strange structural condition as much as possible into the form of steps 5 ft. wide and as long as possible on which to place a concrete blanket or mat before the grout holes were drilled through which to grout the underlying strata.

It was originally anticipated that drilling holes 20 ft. apart and 20 ft. deep would be all that was required satisfactorily to grout the foundation and prevent leakage through the limestone under the cutoff wall, and the proposal included 3,300 lin.-ft. of drill holes. However, the first few holes indicated that this spacing would not be sufficiently close, and finally 10,465 lin.-ft. of holes were drilled.

The process of construction was to open up the cutoff trench with the shovel to rock and then follow this with a crane and clamshell, taking most of the rest of the earth material out of the larger pockets in the rock, then cleaning the rock by hand, loading the material into skips together with the loose stone. This was followed by drilling and shooting the rock peaks, to prepare the ledge in steps for the concrete mat, then placing the mat and a low concrete cutoff wall through which extended 4-in. pipes to be used for drilling, then drilling the holes and grouting, and finally backfilling the trench with suitable rolled material. These various processes were all going on at the same time, one following the other.

It was soon found that the drill holes should be not over 10 ft. apart, and this was tried for some time, but it later developed that 5-ft. holes, drilled in between the 10-ft. holes, often took a large amount of grout. Therefore, shortly after getting under way, 5-ft. holes were the standard; the 20-ft. holes were drilled first and grouted, then the 10-ft. holes, and lastly the 5-ft. holes. The first few holes took 4 to 10 bags, and occasionally a hole would take up to 100 bags; conditions didn't look as if they would be very bad, but, as the job progressed eastward, the holes began to take more and more grout, and nearly every day a record was broken; first 300, then 500 bags per hole, then 900, until at one point a hole took 4,535 bags of cement. It was in this area that the heaviest grouting took place; 10 of the largest holes took over 1,300 bags each, which gives an idea of the caverns and crevices that must have existed below ground.

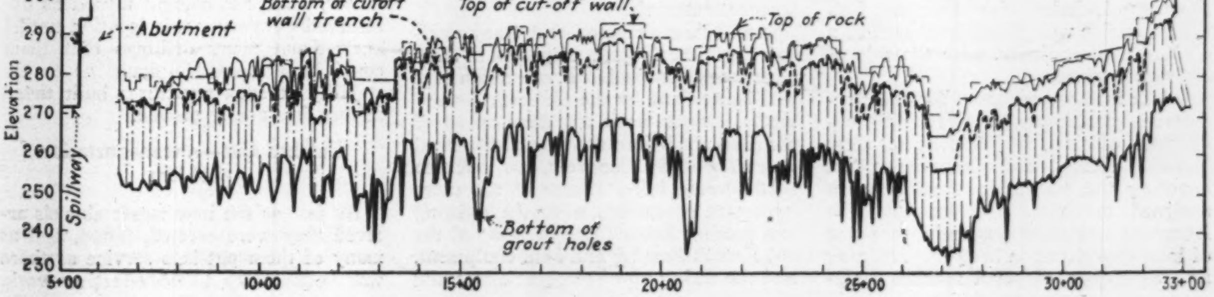
Toward the east end the rock seemed to have fewer crevices and took less and less cement. The original estimate had called for 4,200 bags of cement to be used for grouting. The final estimate paid for 61,368 bags, or about fifteen times as much as was originally contemplated. The grout alone for these holes cost the city almost as much as the amount of the original total bid on the contract.

The grouting plant consisted of one portable air compressor, 310 cu.ft., which was moved along the trench and furnished air to operate the grout machines. Three grout machines were used, mounted on movable platforms which operated along the side of the trench. The discharge pipe from the grout machine was connected to the grout pipe with flexible high-pressure air hose.

The grout pipes, which were about 18-in. long, were inserted in the diamond-drill holes and were fastened in place by first placing a calking link of oakum and then filling around the pipe with cement. This was done immediately after the drilling of the holes was completed, so that the cement was set up ready for the grouting pressure the next morning. If, in case of emergency, which happened very seldom, we wished to grout a hole the same day that it was drilled, we used Mineralead instead of cement for sealing the connection. The top of the 18-in. pipe nipple was threaded to take the standard coupling that was attached to the above-mentioned grout holes. Pressures throughout the job varied from 20 to 30 lb., depending upon the depth of the hole.

It is always a difficult thing to determine, for estimating, just how close together drill holes ought to be placed to grout underlying rock properly. It is also almost impossible to tell how much grout per foot of hole should be allowed for in the estimates. In ordinary rock, sandstone, shale, conglomerates, granite, etc., it is reasonably easy to feel that one's estimate will be somewhere near right because of experience in other structures

FIG. 4—PROFILES along dike, showing rock surface, corewall elevations and depths of holes for grouting the underlying limestone ledge.



in similar foundations, but with limestone all experience, rules and textbooks are likely to be useless. One hole, like the largest one found on the Ontelaunee Dam, which took 4,536 bags of cement, can throw all estimates into the ashcan. This hole alone took more than the total amount of grout figured on for the entire job.

When working in limestone, one is liable to encounter large caverns, wide cracks and water channels in the underlying rock. Perhaps if there are any lessons to be drawn from this job, they are that, in exploring to determine the location of rock in limestone areas, borings and test pits should be taken much more frequently than in country where the rock is not limestone. Second, it would be well to drill into the rock and make tests to determine the amount of grout required by several holes in different sections of the rock before attempting to make up an estimate.

A reservoir flooding a limestone area is not often selected if there is any other place to store water. It would then have been a sad situation, indeed, if this dam had been undertaken by a small community that had only about $80,000 to spend on it. In this particular case, the project is of sufficient size to absorb this overrun, and the other contracts are running very close to the original bids.

The job was shut down during February and March due to the impossibility of completing a small amount of final grading work that remained to be done on account of freezing weather. Work was resumed April 1, and the contract was completed April 3. The structure was not needed, and therefore it was decided to let it lie over the winter. But notwithstanding the greatly increased quantities, the job was to all intents and purposes completed within the contract time.

The engineering work on this job was handled by the staff of the bureau of water under the direction of A. R. O'Reilly, chief engineer.

An Engineer's Recollections—XI

EARLY DAYS AT PANAMA

By **JOHN F. STEVENS**

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Construction Plant and Transportation Service

Construction planning centered around disposal of excavated material as fast as it was dug—Design of the locks and supply of rock pushed—Changes made in administration of the canal work

THE PART PLAYED by civilian engineers in supplying services that were essential adjuncts to the actual construction of the canal, such as feeding, housing, sewage disposal and water supply, was discussed by Mr. Stevens in the preceding article of this series (*ENR*, Aug. 22, 1935, p. 255). The following article, last of the group of three articles on the Panama Canal, discusses the more direct contribution of those engineers to the actual construction operation. —EDITOR.

THE TECHNICAL and other activities of the civilian engineers on the Panama Canal during 1905-1907 were many and varied, but the relation of them here must necessarily be much condensed.

The line of the proposed canal—for purposes of engineering supervision from deep water in Limon Bay to deep water in the Gulf of Panama—was divided into three parts: the Atlantic, Central and Pacific Divisions. The Central Division included the Culebra Cut. Each division was placed in charge of a civilian civil engineer, with the title of division engineer.

These men were chosen for their respective positions, as were all subordinate officials, for reasons of their known ability and experience in handling the particular kind of work assigned to them. For example, the Atlantic and Pacific engineers were men of long experience in handling dredging and all classes of small marine equipment. The selection of the personnel of their staffs and manner of their organization was left entirely in their hands. On my personal staff I had two civilian civil engineers, the duties of the one being to exercise general supervision of the movement of canal trains and to act as inspector of all classes of construction. The duties of the other will be noted later.

Important engineering preparations

After the center line of the canal was established on the ground, engineers were sent into the jungles in charge of field parties; they located with precision the delimiting lines, five miles from, and parallel to, the center line of the canal, between the Canal Zone and the Republic of Panama, marking these lines with concrete blocks set in the ground at frequent intervals.

It will be understood that many engineers were at all times employed in technical work involved in construction that was being done by the different departments. Much work was done on all of the divisions before the type of the canal was adopted by Congress, too much to particularize fully. Some of the more important operations were the refitting and putting into service of some of the old French floating and land equipment, and the fitting up of machine shops and small drydocks. Dredging was done along the common line of whatever type of canal was adopted. New dredges were bought for the Atlantic Division, and old French dredges that could be made relatively efficient were reconditioned and temporarily put in service on the Pacific Division.

Other important work done on both of these divisions included the drilling of test holes to prove the suitability of the foundations for dams and locks, and not until this absolutely fundamental question was affirmatively answered was the location of any of these structures finally chosen. Many hundreds of deep drillings, core and wash, mostly the former, were made at Gatun before we decided upon the location for the immense dam and locks, in case the lock type was to be built. And, as chief engineer, I did not commit myself as unreservedly favoring this location until I knew from many drillings that there could be no possible doubt as to the stability of such structures built there, as time has amply proved.

Culebra Cut—a transportation problem

As fast as the new steam shovels arrived they were erected, tested, and as many of them put into service as there was opportunity to do effective work.

One of the first orders that I issued after arriving on the Isthmus stopped all the merely colorable work then in progress on Culebra Cut, and it was not resumed until after proper preparations had been completed, although I was severely criticised by some of the newspapers for not "making the dirt fly" at once on my arrival. The critics probably would not have recognized a steam shovel if they had met one on the street.

Removal of the material of Culebra Cut, like that of all excavations, large or small, was a problem of transportation; this statement can be accepted as an elementary fact, and it matters not whether the material is moved ten feet or ten miles. To get the maximum efficiency out of any loading machine, steam shovel or dragline, the boom must be kept swinging every possible minute of the time. And this can only be accomplished by keeping empty cars, or trucks, always at hand to receive their loads from the machines. In this case it was wholly a matter of steam railroad operation.

All of the material in the cut was waste, and it had to be hauled long distances to the several places of disposal, by joint train movements over the tracks in the cut and the main track of the Panama Railroad. And, as the excavating, loading and disposal of the immense mass was one of the limiting factors in point of time of the completion of the canal, it was necessary that the most effective system of handling it that could be devised be worked out and adopted, a plan flexible enough to enable it to be expanded as excavation proceeded and the cut became deeper and wider.

The track yard in the cut

A simple but extensive, comprehensive and flexible system of railroad trackage, over which the cars to serve the shovels could be handled with a minimum loss of time, was designed by myself personally and installed according to my plans. It proved the success which I knew that it would, and was continued in use until the material in the cut was entirely removed. It connected with the main track of the Panama Railroad at convenient points, and it was probably the greatest single factor in the nature of a machine that contributed to the success of the construction of the canal.

The division civilian engineer had on his staff an experienced operator as superintendent of steam shovels, and a trained railroad operating man as superintendent of the cut trackage system, and they worked in close cooperation. The movement of the trains over these tracks and those of the Panama Railroad was conducted with little undue delay, and as a consequence the maximum daily output of the shovels was very satisfactory, so much so that by the middle of 1907 it was gradually creeping up monthly to nearly a million cubic yards. A strong spirit of friendly rivalry prevailed among the shovel crews as to which could load the most cars daily.

The work of civilian engineers

The actual practical supervision of the work of excavating and disposing of the material in Culebra Cut was that of civilian engineers, then, and it was so until the completion of the canal.

As an example of the close supervision which the engineers gave to this work: current reports were made to me of the number of cars that each shovel loaded daily, and the loading in cubic yards. The total of this yardage at the end of each month, compared with that reported as the result of careful instrumental cross-sectioning, gave an average difference of less than 3 per cent.

Two large air-compressing plants were installed at equal distances from the ends of the cut and from each other. From these plants the compressed air needed to feed the hundreds of drills was fed into a pipe line extending nearly the whole length of the nine-mile-long cut. While nine-tenths of the material in the cut was comparatively soft rock, it all had to be drilled and shot with dynamite, as there was too much rain to use black powder.

There was no rock suitable for aggregate for the immense amount of concrete to be used in the Gatun locks near the work. A bluff of rock which filled the specifications, located in Panamanian territory some 25 miles down the coast from Colon, was leased from that government. A quarry was opened, equipped with large gyratory crushers, from which crushed rock could be spouted directly into barges alongside, to be towed by tugs up through the reconditioned old French canal to Gatun.

Two large seagoing dredges were, in their general features, designed by the ocean division engineers. They were built later at Baltimore, and sailed under their own power, one to Colon and the other around South America to Panama. The latter, after arrival there, was conditioned, put into service and made the remarkable record of moving more than half a million cu.yd. per month.

Speeding up on the lock canal

Before the type of the canal was decided, and after the trackage system had been installed in Culebra Cut, the work of excavation there was going speedily ahead. But after the type was adopted things began to quicken everywhere. The site of the big dam at Gatun was cleared, the locks were definitely located, and the work of excavation for them began. The same at Pedro Miguel, and at both places the buildings needed for construction purposes were erected. The location of dam and locks at Miraflores was not definitely fixed at that time, as the test borings had not gone far enough to make a final decision. The elaborate and successful mechanical plant which was installed to handle the material for the construction of the Gatun locks was designed and erected by the civilian division engineer.

A force of experts in the designing of locks was organized under the direction of one of my assistants (himself a civilian engineer of experience in lock construction and operation), **which** began the work of designing the immense locks and gates as they were afterward built. At many points along the line active work was inaugurated and pushed with vigor.

The oft-repeated statements that the administration of civilian engineers only covered the preparatory work of building the canal are not true. The real construction of the canal was proceeding actively, and much had been accomplished when that administration came to an end.

The principles of organization

To touch again upon the important matter of an organization to build the canal, I quote a letter written by me:

> Culebra, Aug. 5, 1906
>
> Sir: In compliance with your instructions that I outline my ideas as to a proper organization that will permit the construction of the canal to be carried out in the simplest, quickest, and therefore best manner, I have the honor to report that I have given much thought to the matter, and to say:
>
> I believe that the power and responsibility should be concentrated, not divided; that the commission, constituted in whatever way it may be, must practically be a unit, and as such must resolve itself into a one-man proposition.
>
> That from now on, everything should be made subordinate to construction, and that, complying of course with the law governing, the members of the commission should be such men as will be in direct charge of and responsible for the important phases of the work, and so far as consistent they should live on the Isthmus.
>
> That so long as a large commission must be maintained, the division of duties to each other should be clearly defined, and the chairman should be the responsible head.
>
> That purely governmental functions should be entirely separated in theory and practice from the work of canal building, excepting so far that all officers representing the United States shall understand that the sole and only object they have in holding office and living on the Isthmus is to enable the United States to build and operate the canal.
>
> I have, in consultation with Chairman Shonts, thoroughly threshed over these matters, and wish to say I am entirely in accord with him, and that I believe an organization along the lines we have drawn up, a draft or outline of which he takes to the States with him tomorrow, is the best and strongest that can be devised.
>
> While the canal may be paid for by the eighty-odd millions of the people of the United States, the construction of it can be successfully carried on only under the supervision of a very limited number of them.

The "threshing over" mentioned was done after I had written the letter, which was not changed by a dot or a dash. I consulted with no one in formulating it. For reasons of policy which at that time seemed best to maintain, I phrased it as I did. It will be noted that there is a striking similarity in the suggestions that I outlined and the plan which the President put into effect at a later date.

Railroad operation and relocation

I have not touched upon the Panama Railroad, which was under my general supervision as being a fundamental necessity in the construction of the canal. A manager was appointed who was a master of detail in all matters of operation. The officials worked in perfect coordination with those of the canal, and with its comparatively heavy commercial traffic and burdened with the many canal trains, its operation was remarkably well conducted. Such parts of it as carried the great bulk of the canal "waste" trains were double-tracked and provided with signals, and other needed physical improvements were made in many places.

These improvements were planned and supervised by its civilian chief engineer and his assistants, as also was the technical work and supervision of the locating and constructing of its new line made necessary by the creation of Gatun Lake, whose waters now submerge nearly the entire mileage of the old line. The operation of the railroad and the conducting of the vast and intricate work on the canal were virtually interlocked, and the deep interest which the officials took in the work made the perfect teamwork which produced results.

It seems entirely unnecessary to particularize the many details which were planned and installed in connection with the few major operations noted, or to describe more of the latter class. I can only say that wherever temporary or permanent work could be done it was undertaken and vigorously pushed, and the results of the work done 1905-1907 were very satisfactory to me—and, I will add, to some other interested parties.

Civilian engineers and General Goethals

It probably will be assumed by some that my account of what the civilian engineers accomplished during that period is colored by the fact that I am an *ex parte* witness. At the risk of being considered very conceited (my friends know better, others are entitled to their own opinion), I am going to add the testimony of one who was in a position to know the exact condition of the work when I resigned. It is a voice from the great beyond.

When the untimely death of General Goethals in January, 1928, was announced, there appeared in the press many reminiscences of him and his career. I quote verbatim from the New York *Evening World* of Jan. 24, 1928:

> With the passing of Major-General George W. Goethals, the inside story of the building of the Panama Canal can be told—that is, General Goethals' own account of certain things that made accomplishment of his task possible. Of the incidents related here General Goethals spoke only in confidence, and with the understanding that his remarks were not to be published at the time.
>
> General Goethals never boasted of his great accomplishments, and when the Canal was mentioned in his presence he always insisted that two men, Theodore Roosevelt and John F. Stevens, had far more to do with the successful building of the canal than he. He had followed Stevens as chief of the work of construction, and his admiration for his predecessor was evident at all times.
>
> Stevens, he would say in his quiet way, was one of the greatest engineers that ever lived, and the Panama Canal is his greatest monument. He was a wonderful organizer and a remarkable judge of men. He had unerring insight in the selection of his assistants, and I found when I went to Panama that his organization was about as perfect as any one could make it. The result was that more than one-half of the work was done for me in advance.

At other times and on other occasions General Goethals said things that were even more impressive. That his feeling of "admiration" carried him in his language beyond justifiable limits is of course apparent. But he comprehended and appreciated the vast and valuable work which was done by the engineers on the canal 1905-1907; as to this there can be no difference in opinion. And that he expressed his appreciation of it in such glowing terms demonstrated the magnanimity of his character. General Goethals and I were friends and understood each other thoroughly, and I mourned sincerely his early passing.

A well-planned and well-built machine

We handed over to the army engineers a well-planned and well-built machine, which perhaps ran at first with a squeak or a hot bearing here and there, as is always inevitable with new machinery, but which soon found itself and ran, comparatively speaking, like a high-grade watch. The practical proof of this is that it operated with no radical changes until the completion of the canal.

The work of General Goethals and his colleagues in the building of the canal, was, in my judgment, above just criticism. The manner in which the general plans were rounded out and the installation of the great mass of necessary details have always excited my admiration and respect. It is of course certain that civilian engineers could have been found who would have done equally as well, but that they could have done better I do not believe. And in expressing my opinion of the excellent work done, I am not forgetting that many civilian engineers had prominent parts in its execution. The record of the army engineers on the canal, from the time they took charge in 1907 to the time of its completion, upheld the finest traditions of that famous corps, and the administration of affairs on the Canal Zone, including the operation of the canal, has been a marked success.

The inevitable but inane question arose after the completion of the project —"Who built the Panama Canal?" The engineers of the United States, regardless of titles or classification, built the canal. It is a marvelous working entity, a symbolic expression of the genius and resourcefulness of the profession, as being the dominant factor in the material progress of the world.

An engineer's specifications

When I received the first offer to become chief engineer of the canal, I declined it. Then I was asked to me William Nelson Cromwell in New Yor which I did, and after listening an hou or two to his silver-tongued argumen I consented to accept the position—wit conditions. But just before sailing t the Isthmus, I was, with the chairma of the commission, bidden to visit th President at Oyster Bay. At this conference I told the President plainly tha I had accepted the position against m real wishes; that I was a man of fe words and that those might be too blun for the occasion, but I found that wa what he wanted and apparently liked So I repeated the conditions of my acceptance which I had made with Mr. Cromwell and which were approved by the chairman.

They were: That I was not to be hampered or handicapped by anybody high or low in my attempts to straighten matters out; that I must be given a free hand or none at all; that while I quite understood that I was technically to report to the commission, the organization, relations to the governor of the canal zone, relations with other departments than that of engineering, selection of plant, material and supplies, forces of skilled and unskilled labor, care of and handling of the same, must come under my undisputed jurisdiction; that I could not promise to put the canal construction on a proper basis, but that every last effort of which my mind and body were capable would be bent to that end.

I also told him that I had made and could make no promise to remain until the work was fully completed, but that I would stay with it until I had made its success certain, or had proved it to be a failure, and that I should consider myself free to exercise my own judgment in both matters. He raised no objection to my statements, tacitly approving them, simply saying that things were "in a devil of a mess down there" and that he depended upon me to make the canal a success, which it must be without fail. He gave me no orders but told me to feel free to consult with him in any or all matters. He carried out his promises to the very letter and supported me with all his vast power. His cutting of corners and red tape was a joy. His methods gave me great confidence, and my relations with him were very harmonious during my term of service and continued so after it had ceased.

Success assured by 1907

Various reasons for my resignation were given by irresponsible scribblers. They all had points of similarity, as they were all stupid and mendacious. In one respect they were exactly alike; they were all absolutely untrue. I resigned for purely personal reasons, which were in no way, directly or indirectly related to the building of the canal, or with anyone connected with it in any manner. I knew better than anyone else that the work was so well organized, planned, equipped and initiated, that there could be no doubt but that competent engineers, administrative and constructive, could be found to carry it on to a successful completion. If I had not known all this, I would have remained as chief engineer. I could have done so and had full powers under the plan of organization which the President put into effect, and to which I have previously referred. My totally unexpected appointment as chairman of the commission, and later official intimation that I received, are ample proof of this statement.

To summarize results as they existed in the middle of 1907: The successful completion of the Panama Canal at that time, barring the occurrence of some cataclysm, was as well assured as it was on that historic day, Aug. 3, 1914, when the first ocean steamship passed through it from the Atlantic to the Pacific. With the menace of yellow fever removed, it was the work of civilian engineers that in 1907 gave assurance to the world that its dream of centuries of an Isthmian Canal would become a reality.

Beam-Strengthening Effect of End Connections

Laboratory investigation of beam-to-column connections demonstrates appreciable stress reductions possible by taking into account continuity or restraints at beam ends

By Kenneth G. Merriam
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TO WHAT EXTENT do riveted column connections "strengthen" a steel beam? This question has been one of real interest to engineers and designers in recent years. Although it is commonly known that restraining couples applied at the ends of a beam will reduce the algebraic value of the bending moment at all portions of the span, a common design procedure for riveted steel connections has been to assume that such restraining couples do not exist. One reason for such an assumption is that too little information has been available relating to the magnitude of the restraining moment to be expected in such cases.

In 1931 the writer was approached by several engineers who suggested that an attempt be made to obtain laboratory measurements relating to this problem. A method of attack was evolved, and the laboratory work was then undertaken by John C. Osipowich and George M. Hansen, as a student research project, and general supervision of the work was assumed by Marion L. Price and B. Leighton Wellman, members of the mechanical engineering instruction staff of Worcester Polytechnic Institute. Materials and advice were generously furnished by the Eastern Bridge and Structural Co., of Worcester, through Lester W. West.

From the experimental work, carefully and thoroughly executed, results were obtained that were of more than ordinary interest to all concerned. Through a peculiar combination of circumstances, however, no effort at that time was made to publish the results. After an interval of four years, therefore, the present attempt is made to abstract the findings of Messrs. Osipowich and Hansen.

For additional data on the same question, reference may be made to the following, published both before and since the investigation reported here was carried out: "Rigidity of Joints of Steel Structures," by W. M. Wilson and H. F. Moore, Bulletin 104, University of Illinois Experiment Station (1917). Second Report, Steel Structures Research Committee (London, 1934) by Prof. Cyril Batho; "The Relative Rigidity of Welded and Riveted Connections," by Prof. C. R. Young, Canadian *Journal of Research*, July and August, 1934; and "The Elastic Properties of Riveted Connections," by Prof. J. Charles Rathbun, *Proceedings*, American Society of Civil Engineers, January, February and May, 1935.

Basis of the investigation

The technical basis of the investigation is elementary and can easily be understood by brief consideration of Fig. 1 (left), which shows a schematic diagram of the loading arrangement. If a continuous beam, 20 ft. long, is freely supported at the ends S and N, and also by a support at the mid-span point C, which is on the same level with the end supports, and if the beam is loaded by two equal concentrated loads at "quarter-points" (accomplished by using a distributing beam, as shown), then the end reactions will be equal, the elastic curve of the entire beam will be symmetrical with respect to the mid-span point C, and the slope of the elastic curve at this point will be zero.

To obtain the bending-moment diagram for the perfectly continuous beam, the portion CN may be considered, using the principle of superposition. The upward deflection at N produced by the reaction R_1 (see Fig. 1, right), acting alone, must be equal to the downward deflection at N produced by the force $P_t/2$, acting alone. From this, it follows that $R_1 = \frac{5}{32} P_t$, $R_2 = \frac{11}{16} P_t$, $M_C =$

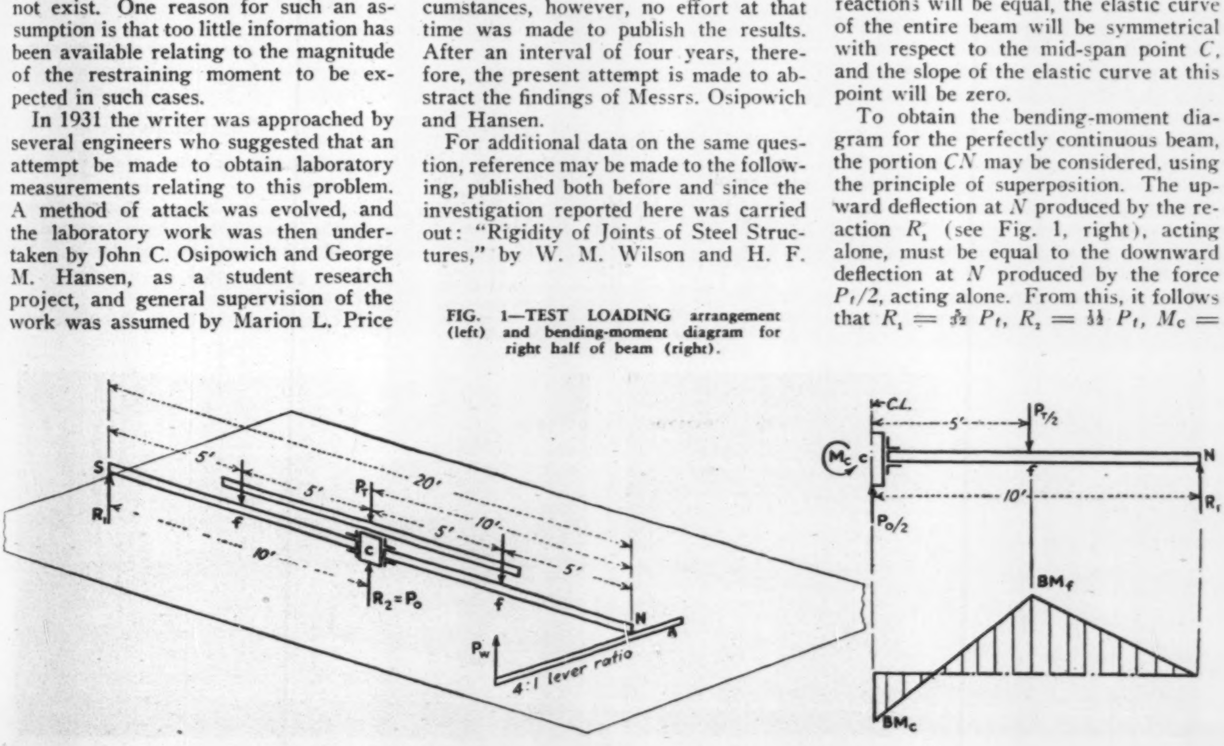

FIG. 1—TEST LOADING arrangement (left) and bending-moment diagram for right half of beam (right).

$\frac{15}{16} P_t$ (ft. = lb.), and $M_f = \frac{25}{32} P_t$ (ft. = lb.). From Fig. 1 (left) it is evident that $P_t = 2R_1 + R_2$, so that

$M_c = -\frac{15}{16} (2R_1 + R_2)$ and $M_f = + \frac{25}{32} (2R_1 + R_2)$

for the contiunous beam case.

In any case,

$M_c = 5R_1 - \frac{5}{2} R_2$ and $M_f = 5 R_1$.

The above reasoning shows that, if a specimen of beam-to-column connection is introduced at C (see Fig. 1, left) and if all supports are kept on the same level, it will be possible to obtain the bending-moment diagram for the actual case and also for the case of ideal continuity at C, simply by measuring the values of R_1 and R_2.

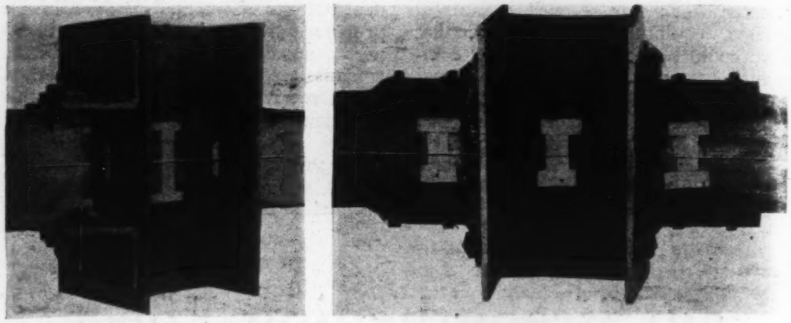

FIG. 2—TYPES of beam-to-column connections investigated for the strengthening effect they impart to the beams.

"Per cent continuity"

The investigators used a term which they called "per cent continuity" and which they defined as follows:

At any given section of the beam, "Per cent continuity"

$$= 100 \frac{\text{actual bending moment}}{\text{ideal bending moment}}, \text{ where}$$

"ideal bending moment" was understood to mean the bending moment that would occur at the given section if the beam were perfectly continuous at C. Under this definition, then,

per cent continuity at C =

$$100\left(\frac{8R_2 - 16R_1}{3R_2 + 6R_1}\right)$$

and per cent continuity at f =

$$100\left(\frac{32R_1}{5R_2 + 10R_1}\right)$$

Degree of tightness

Also used by the investigators was a term which they called "degree of tightness," the definition of which they developed as follows:

Let θ be the angle through which a nut is turned in tightening a bolt from "finger-tight" condition to "full tightness," which is the tightness produced by a man of average strength using a wrench with a handle 15 in. long, and applying full effort.

The "degree of tightness," then, corresponding to $K\theta$, is defined as the value of K.

In the words of the experimenters, "The basing of the degree of tightness upon the fractional part of the number of turns needed to draw each bolt fully tight was the most accurate method we had to measure easily how tight the connections were."

Evidently the "degree of tightness" can vary from zero to unity, and is only a rough index of "tightness," even if the same man is to execute all tightening operations. It is, however, useful and adequate for the case at hand.

Object of the investigation

Having discussed the technical elements and definitions involved, it now becomes feasible to state the technical object of the investigation, as follows: the use of full-scale structural elements, to find the relation between per cent continuity and degree of tightness for each of two common types of beam-to-column connections, shown in Fig. 2.

Test apparatus and results

The test arrangement can be understood by referring to the illustrations. A 12-in., 25-lb. I-beam was used for the distributing beam. The beams that were connected to the column were 6-in., 12.5-lb. I-beams. The short length of column used was an 8-in. H-section.

The reaction R_2 was measured directly (Fig. 3, left) by a 400,000-lb. testing machine, and was sometimes called P_c. The reaction R_1 was measured indirectly by another testing machine, using a 4-to-1 lever as shown in Figs. 1 and 3, right, and the load measured by the testing machine was called P_w.

Because of the necessity for vertical adjustment of point N (Fig. 1) in two planes, a spherical seat was used to connect beam and lever at this point (Fig. 3, right). To keep the points S, C and N (Fig. 1) on the same straight line, a jack screw was provided at S (Figs. 1 and 3, left) by means of which the point S could be raised or lowered as indicated by a loaded wire, stretched between S and N, and paper scales attached at C (Figs. 2 and 3). This jack-screw load was not measured at all, conditions of symmetry making it reasonably certain that the loads at S and N must be equal.

From Tables 1 and 2, which list data and results, the sequence of operations is apparent. A graphical representation of the results is given in Fig. 4.

Conclusions

Significant conclusions may be listed as follows:

1. For a given type of bolted connection, the restraining couple, exerted by the column on the end of the beam, is roughly proportional to the degree of tightness of the bolts.

2. For two given types of bolted con-

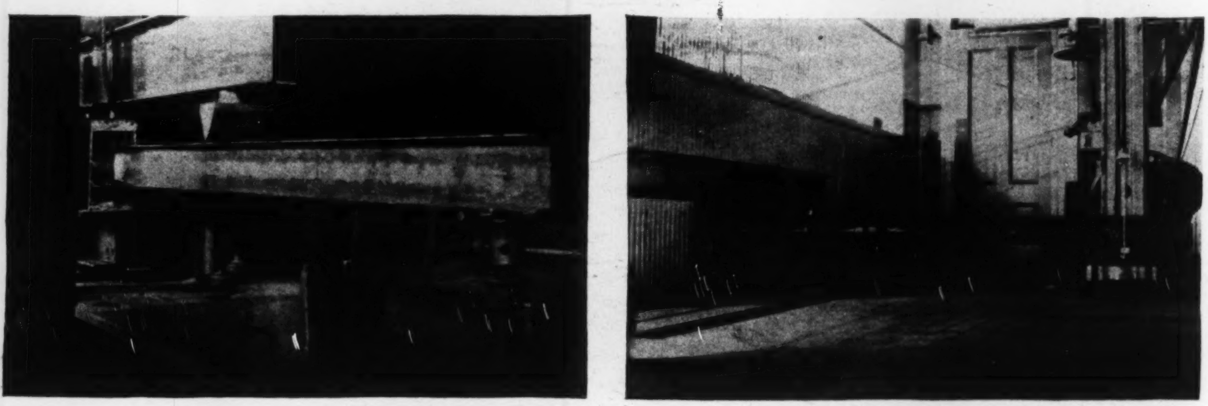

FIG. 3—VIEWS of test set-up, showing (left) loading arrangement at S and C of Fig. 1 and (right) loading arrangement at N, together with the lever and the testing machine that measured its load.

nections, having bolts of equal size, number and tightness, the greater restraining couple, exerted by the column on the beam, will be furnished by the connection that is made with the more rigid connecting elements.

3. For a given type of bolted connection, with bolts fully tight, the "degree of continuity" is independent of the applied load, so long as no part of the structure is stressed beyond the yield point.

4. If, for the case at hand, rivets can be considered equivalent to bolts that are drawn fully tight, then the angle type of riveted beam-to-column connection (Fig. 2) may furnish as much as 55 per cent of continuity at the connection. For an I-beam that is supported at the ends and loaded with a uniformly distributed load, 50 per cent continuity is as effective in enhancing strength as 100 per cent continuity would be. The maximum bending moment in such a case would be reduced from $wl^2/8$ to $wl^2/12$, which means that the maximum stress in the beam might be reduced 33 per cent by the presence of the connection.

5. If, for the case at hand, rivets can be considered equivalent to bolts that are drawn fully tight, it is probable that the windbracing type of connection, which resembles that shown in Fig. 2, right, may produce as much as 90 per cent of ideal continuity at the connections.

6. The beam-strengthening effect of beam-to-column connections is not directly proportional to the per cent continuity at the connections. For the illustration mentioned under item (4) above, for example, the maximum strengthening effect would be obtained with a per cent continuity of 75, under which condition the maximum positive and negative bending moments would each be $wl^2/16$, and the stress in the beam would be reduced 50 per cent by the presence of the connection. The strengthening effect, for a given per cent continuity at the connections, then, depends upon the nature of the loading.

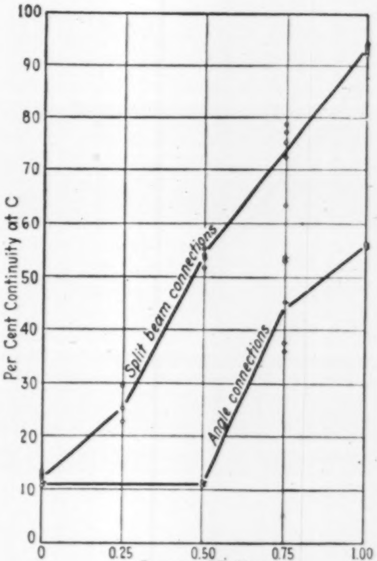

FIG. 4—RELATION between degree of tightness of bolts and per cent continuity at *C*, Fig. 1.

TABLE 1—DATA AND RESULTS FOR STRUCTURAL ANGLE CONNECTION

P_e Lb.	P_w Lb.	P_t Lb.	Degree of Tightness	Actual B.M. in Ft.-Lb.		"Ideal" B.M. in Ft.-Lb.		Per Cent Continuity	
				c	*f*	*c*	*f*	*c*	*f*
5,000	409	8,272	Full	4,320	8,180	7,760	6,460	55.6	126.5
4,500	367	7,436	"	3,910	7,340	6,980	5,805	55.8	126.2
4,000	327	6,616	"	3,460	6,540	6,220	5,160	55.6	126.6
3,500	286	5,788	"	3,030	5,720	5,420	4,525	55.8	126.5
3,000	244	4,952	"	2,620	4,880	4,630	3,875	55.8	126.2
2,500	204	4,132	"	2,170	4,080	3,880	3,225	55.9	126.9
2,000	164	3,322	"	1,740	3,280	3,100	2,580	56.0	127.0
1,500	123	2,484	"	1,280	2,460	2,320	1,940	55.2	126.8
1,000	82	1,656	"	860	1,640	1,550	1,295	55.6	126.8
500	41	828	"	430	820	778	647	55.5	126.7
0	0	0	"	0	0	0	0	....	......
5,000	478	8,824	¾	2,940	9,560	8,260	6,900	35.6	138.6
4,000	373	6,984	"	2,540	7,460	6,560	5,450	37.2	138.0
3,000	267	5,136	"	2,160	5,340	4,810	4,015	44.9	133.0
2,000	167	3,336	"	1,660	3,340	3,140	2,610	52.6	128.0
1,000	83	1,666	"	833	1,666	1,565	1,300	53.2	128.0
0	0	0	"	0	0	0	0	....	......
4,000	461	7,683	½ and finger	780	9,220	7,220	6,000	10.8	153.5
3,000	345	5,760		600	6,900	5,400	4,500	11.1	153.3
2,000	230	3,840		400	4,600	3,600	3,000	11.1	153.4
1,000	115	1,920		200	2,300	1,780	1,500	11.2	153.3
0	0	0		0	0	0	0	....	......

TABLE 2—DATA AND RESULTS FOR SPLIT H-BEAM CONNECTION

P_e Lb.	P_w Lb.	P_t Lb.	Degree of Tightness	Actual B.M. in Ft.-Lb.		"Ideal" B.M. in Ft.-Lb.		Per Cent Continuity	
				c	*f*	*c*	*f*	*c*	*f*
5,000	304	7,430	Full	6,420	6,080	6,960	5,800	92.3	104.7
4,500	273	6,685	"	5,790	5,470	6,260	5,220	92.5	104.7
4,000	244	5,953	"	5,120	4,880	5,580	4,650	91.8	105.0
3,500	213	5,205	"	4,500	4,260	4,880	4,070	92.1	104.7
3,000	181	4,450	"	3,880	3,620	4,170	3,475	93.2	104.3
2,500	150	3,700	"	3,250	3,000	3,470	2,890	93.8	103.8
2,000	120	2,960	"	2,600	2,400	2,770	2,310	94.0	103.8
1,500	90	2,220	"	1,950	1,800	2,080	1,732	93.8	103.8
1,000	60	1,480	"	1,300	1,200	1,390	1,155	93.5	103.8
500	30	740	"	650	600	694	580	93.6	103.4
0	0	0	"	0	0	0	0	....	....
0	0	0	¾	0	0	0	0	....	......
1,000	70	1,560	"	1,100	1,400	1,462	1,209	78.6	115.9
2,000	140	3,120	"	2,200	2,800	2,925	2,435	75.3	114.9
3,000	210	4,680	"	3,400	4,100	4,385	3,660	77.4	112.0
4,000	286	6,290	"	4,275	5,725	5,900	4,910	72.5	116.7
5,000	386	8,090	"	4,785	7,715	7,580	6,320	63.3	122.0
0	0	0	½	0	0	0	0	....	......
1,000	82	1,656	"	844	1,640	1,550	1,293	51.5	127.0
2,000	164	3,320	"	1,720	3,280	3,115	2,595	55.1	126.5
3,000	249	4,990	"	2,520	4,980	4,680	3,900	53.7	128.0
4,000	333	6,665	"	3,340	6,660	6,240	5,200	53.5	128.0
5,000	416	8,330	"	4,190	8,310	7,800	6,500	53.8	127.9
0	0	0	¼	0	0	0	0	....	......
1,000	100	1,800	"	500	2,000	1,687	1,406	29.7	142.3
2,000	200	3,600	"	1,000	4,000	3,375	2,810	29.6	142.3
3,000	311	5,490	"	1,285	6,215	5,150	4,280	25.0	145.2
4,000	422	7,475	"	1,560	8,440	7,010	5,840	22.3	144.2
0	0	0	Finger	0	0	0	0	....	......
1,000	113	1,904	"	240	2,260	1,785	1,489	13.4	152.0
2,000	227	3,816	"	460	4,540	3,570	2,975	12.9	155.0
3,000	343	5,744	"	640	6,860	5,380	4,480	11.9	143.0
4,000	358	7,664	"	840	9,160	7,200	5,980	11.6	142.2

7. To determine whether bolts drawn fully tight can be considered equivalent to rivets, an obvious step would be to extend the test to include an actual riveted connection. The test procedure is very simple to execute, and it would seem that further investigation might be warranted.

8. Proper application of information resulting from this investigation, and others of like nature that may follow, should make possible more rational design and lighter riveted steel structures.

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Lumber Supply and Demand Nearly Balanced

Lumber supply and demand are reasonably balanced for the first time in years, and the Commerce Department's lumber survey committee, for the first time in its history, does not recommend restriction of production, but notes a heavy reduction in stocks and an increase in consumption on a rising price trend.

Shipments during the first six months of 1935 were approximately 12 per cent above production, and new business was 18 to 20 per cent above output. Despite the closing of many small mills, 1935 production is expected to reach 15,500,000,000 b.ft. and consumption to reach almost 17,000,000,000 b.ft. Consumption during the first half totaled 8,000,000,000 b.ft. Stocks on July 1 totaled 7,000,000,000 b.ft., a reduction of 10 per cent since January.

Letters to the Editor

Rotary Caisson-Sinking

Sir—In *Engineering News-Record* of July 11, 1935, there appears an article entitled "Rotating Caissons Drill Their Way To Rock," which is illustrated on the cover and editorialized on p. 32, and which refers to the method and apparatus used in the installation of the caisson foundations for the new Federal Office Building at Vesey St. and West Broadway, New York City.

This matter gives the impression that the principles of that method and apparatus were first used commercially on that particular job, whereas in our opinion they were developed by others and are protected by patents now owned by the Caisson Contracting Co., of New York City.

Prior to that particular job, preferred forms of the method and apparatus had been employed by the Caisson Contracting Co.; for instance, in the glass plant, steel mill, open hearth and pressed steel buildings of the Ford Motor Co.'s Rouge Plant, Dearborn, Mich. and an addition to Building 83 of the General Aniline Works at Rensselaer, N. Y. Those contracts alone involved the installation of 84 caissons under a variety of conditions, including cramped working space, limited headroom, adjacent structures on shallow foundations, quicksand and boulder formations, and depths up to 110 ft. Moreover, all of those caissons were installed by means of hydraulic rotary drilling in which the lower end of the caisson cylinder itself served as the drilling tool. All the cylinders concerned, with three early exceptions where the pneumatic method was used in the final bottoming, were drilled to a seal in rock or in hardpan overlying rock so that the caissons were bottomed on rock in the open air.

Furthermore, the Caisson Contracting Co. installed the first caisson (No. 82) on the Federal Office Building job itself for the George J. Atwell Foundation Corp., which employed the former to do so when the latter procured the subcontract for the foundations of that building. This caisson was installed by the Caisson Contracting Co. with one of its rotaries which had been used on its prior work and without recourse to the pneumatic method, and the caisson was approved by the government. The remaining caissons were thereafter installed by the Atwell corporation with the apparatus illustrated and described in your issue of July 11, which, although different in form from that used by the Caisson Contracting Co., utilizes the same principles and, in our opinion, is essentially the same in function.

This and certain other modern developments in the art of caisson installation are of such importance to the industry that the publication in *Engineering News-Record* of an article broadly describing their history and particular field would surely be appreciated by your readers, particularly if the article were written by someone in a position to know the facts.

CHARLES L. POWELL,
President, Caisson Contracting Co.

New York, N. Y.,
Aug. 29, 1935.

Mass Changes in Member Grades of Am. Soc. C. E. Opposed

Sir—Herewith please find a letter which I hope wou will be able to print in an early issue of *Engineering News-Record*. The most logical place for such a letter would be in *Civil Engineering*. However, a ruling of the publications committee of the Am.Soc.C.E. of some years standing apparently closes the columns of that magazine to discussions on the part of the membership of society affairs. As the most widely read organ in the civil engineering field, *News-Record* seems to me the next most logical place for a letter of interest to Am.Soc.C.E. members.

Sir—The membership of the American Society of Civil Engineers now has before it a constitutional amendment (the so-called Proposal No. 1) providing that hereafter associate members be called members and that 86 per cent of the present members be automatically transferred to a new grade of fellow.

As now constituted, the corporate membership of the society is divided into two nearly equal groups. The intent of the sponsors of this new proposal seems to be an ultimate regrouping so that perhaps three-fourths will be in a lower group (members) and the remaining minority in a more select group (fellows). This is a perfectly legitimate proposal and one that may be in the interest of the membership. But to attain the end of limiting the higher grade of membership to "those notable in the practice of engineering," the amendment proposes a mass transfer of all members who have held membership for five years and who began to practice civil engineering 25 years ago, irrespective of the merit of their accomplishment and even irrespective of whether they have continued in the practice of engineering. The means is so violently opposed to the end that even the committee charged with presenting the merits of the proposal to the membership was forced to recognize that the amendment as framed contains an unworthy bid for votes for which they felt moved to apologize.

In urging the proposal on the membership, the committee makes an eloquent plea, which, however, boils down to about this: "The architects have been able to go places economically and socially denied engineers. This is due to the distinction granted them by their calling themselves 'fellows.' We now propose that the great majority of your members confer this fellowship on yourselves so that you too can go places socially and economically. Irrespective of individual distinction, irrespective of whether you yourselves could meet the requirements which you propose hereafter to impose on those who would join your exalted band, you have but to vote yourselves fellows to open to yourselves those pleasant fields. It is as easy as all that. However, we don't propose that quite all of you become fellows. That wouldn't look right. So we are leaving out *777*, not because they may not have just as solid accomplishments back of them as some of you more fortunate ones. We simply can't transfer the *whole* membership. Those *777* are fine generous chaps. We can't bring ourselves to believe that they will be so ungenerous as to want their cut when they see the pie generally being passed out." In far, far more elegant language, so speaks the committee.

How much more worthy of the past character and dignity of the society is the statement of the committee opposed to the amendment:

> Transfer to any higher grade should be made on merit through the scrutiny of professional records, not by such mass transfers as the amendments propose. Grades so filled can never rise above their origin.

Names have little meaning; what underlies the name is important. If "fellow," as applied to fellows of the American Institute of Architects, carries distinction, it assuredly is not because of any special magic in the six letters composing that word. The designation carries no more weight than the requirements underlying it give it. If we are to have fellows in the society (and it may be very well to have them) let *all* of them achieve the grade in the "normal and natural manner" that the committee in favor commends to the younger members. Then truly the term "fellow" will convey distinction. But as proposed, it means no advancement in standing, nor will it for many years to come. For every interested person will know its genesis.

Similarly, the change of title from associate member to member will carry no added distinction, for all interested persons will know that it is a change merely in name. The requirements for membership hereafter are to be exactly what the requirements for associate membership have been heretofore. There is no promotion here, but quite the contrary.

A very considerable proportion of the present associate membership would normally attain full membership in the next several years. It is now proposed to keep all these men in what will undoubtedly be recognized as *the lowest grade of corporate membership* for an additional thirteen years on the average (the difference between *twelve* years as now required for advancement and the 25 years proposed). During all of this time they will be held on a par with the man who has had five years of practice and one year of responsible charge as principal or assistant. Is the bait worth the price? Is the hollow name of member, so widely to be given, worth the loss of the higher rank of corporate membership which soon would be achieved by so many of these men? (And which would be achieved through their individual merits, and carry a corresponding weight.) How many of our present associate members would not, after thought, prefer to be advanced on their own individual merits to a grade that really stands for something over and above their previous accomplishment? How many wish merely the hollow title without its significance?

And how many of the present members who have really become "notable in the practice of engineering" would not prefer advancement to fellow on their individual merit than to be pitchforked into the grade through a mass promotion? If the proposed amendment passes, in the case of every fellow promoted next Nov. 15, until he dies, this question will remain unanswered: Would he have been elected to the grade on merit, *or did he simply vote himself into it?* How many will enjoy being in that position for the rest of their lives?

To both of these groups the proposed amendment does an injustice. It is still more unfair to the younger members. It wipes out completely the standards to which they submitted themselves and by which they were measured when they were admitted to full membership. It demotes them from the higher to the lower grade of corporate membership, placing them exactly in a par with the man just beginning to attain professional maturity. Many older members, but of no greater accomplishment, are advanced beyond them; many younger associate members of much less accomplishment are made equal with them. Is there justice in this? And does the society need to impose on them this injustice?

If the amendment as it has been drawn were to be subjected to thorough discussion by the membership of the society, I would have no least fear of its adoption. But without such discussion, it is a sort of gold brick that seems to offer what it does not. It really benefits no one, and debate of the proposal would quickly establish this fact. Unfortunately, the established procedure for amending the constitution of the Am.Soc.C.E. does not insure discussion of amendments, and too often they pass by default. Very often a member simply without thought gives his O. K. to an action of the board of direction.

As the committee opposed to this proposal says, "If a different or more consistent set of terms is desired for the membership grades, a discussion should be opened in *Civil Engineering* and local sections should be consulted before submitting amendments for final action with a take-it-or-leave-it gesture." Before this amendment is canvased, meetings of local sections should be called for discussion of it, and at such meetings the attention of members should be directed to the fact that *any votes already cast can be recalled and changed at any time before the vote actually is canvased.*

New Haven, Conn.,
Aug. 27, 1935.

WILLIAM J. COX,
Member, Am.Soc.C.E.

Stiff-Leg Derrick Made From Materials on the Job

BECAUSE the west portal of the Mono Craters tunnel, under construction by the bureau of water works and supply of Los Angeles, is 350 miles from the city where equipment and materials are purchased, it is often desirable to improvise from materials at hand rather than to order special equipment. As a result, there are found on the job numerous instances of the ingenuity of those responsible for the speed and efficiency of the work. A typical example is a 5-ton stiff-leg derrick that is convenient for loading, unloading and transferring heavy freight at the west portal camp. The only parts of this derrick rig that were ordered specially are the four sheaves over which the cables pass. The frame is made up of pipe, as shown in the accompanying illustration; and except for the sheaves all of the parts and materials were found in the stock room, garage, etc., at the camp.

The 16-ft. mast is made of 8-in. pipe. Attached to the mast and rotating with it are the wooden platform for the operator and the two hoists driven by compressed air that operate hoisting line and boom line respectively. There is no bull-ring; the load is swung by pulling on ropes that hang from either side of the boom end.

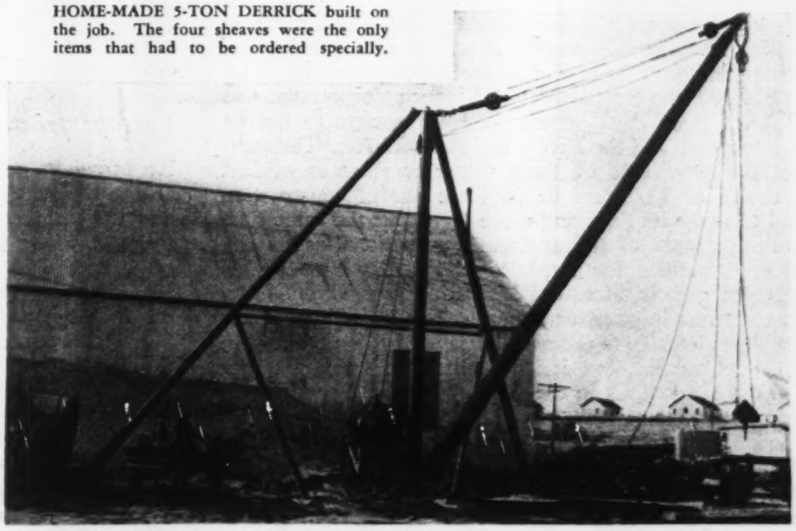
The two stiff-legs that support the mast are 6-in. pipe, and they have one horizontal and two inclined braces of 2-in. pipe. All pipe joints are welded. The two legs and the two stays are bolted at the bottom to steel bars embedded in the concrete piers that were placed to support the derrick members.

Perhaps the most ingenious part of the entire rig is the step bearing at the base of the mast, which makes it easy to swing the mast and boom assembly by hand. The mast footing rotates on a roller bearing of the truncated-cone type taken from an old tractor. The shaft for this bearing was set vertically in the concrete pier and passes through a hole in a ½-in. steel plate laid on top of the concrete and bolted down. Around this vertical shaft was fitted the roller bearing and sprocket salvaged from old tractor parts. The roller bearing was made part of the foundation. The sprocket rotates on the roller bearing and is welded to the base of the 8-in. pipe that constitutes the mast.

At the top of the mast the two stiff-legs attach to a vertical shaft or pin extending down a few inches into the mast inside a collar, which permits the mast to rotate around the pin.

Credit for the design of the derrick rig is given by the engineers in charge of the project to Ed Hendricks, general foreman at the west portal camp.

HOME-MADE 5-TON DERRICK built on the job. The four sheaves were the only items that had to be ordered specially.



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September 5, 1935

Fall Activity

AS SEPTEMBER opens, the summer's currents of business activity, decidedly unusual for the hot months of the year, show every indication of quickening to brisk autumn pace. Not for five years has the general trend at the opening of the fall season been as favorable as now, when good crops and revived initiative give support to the natural growth of business activity. Since the factors of improvement obviously mean much for capital enterprise, it seems reasonable to look for steady advance in engineering construction activity. Meanwhile the maintained rise in industrial-plant expansion, on which we commented two weeks ago, supplements the large amounts of construction that are being brought into action under the new PWA program. The relief operations of WPA and related agencies of the Works Act campaign add an indirect supporting effect by stimulating the materials and equipment industries. Present signs, thus, are distinctly forward, and with the approaching rise of housebuilding from its abysmally low level there is now fair prospect that construction will make steady progress toward putting its large army of unemployed again to work.

Congress and Construction

SURVEYING THE ACCOMPLISHMENTS of the last session of Congress broadly, one is impressed with the fact that engineering and construction have reason to look on them with satisfaction. Many of the major measures enacted reveal that Congress has acquired a new appreciation of the bearing of construction on economic welfare and on recovery. The Works Act is the outstanding expression of this recognition, but further expression is found in half a dozen measures authorizing construction activities either for emergency and recovery purposes or for closely associated public improvements. Among them are the Soil Erosion Act, the Tennessee Valley Authority Amending Act, the Home Mortgage Relief Act with its provision for improvement loans up to $50,000, the Reconstruction Finance Corporation and Disaster Loan Acts, extending RFC powers to finance construction, and the Rivers and Harbors Act. Throughout the provisions of these measures there is evident a desire of Congress to facilitate useful construction and to make its financing more free. At the same time the importance of soil and water as national assets and as opportunities for carefully studied development has received greatly increased legislative recognition. The new interest in construction as well as in soil and water problems led also to the near-passage of a comprehensive flood-control act, but it is fortunate that this measure did not succeed, as in the form proposed it would have had a paralyzing effect on local initiative. At a late session, with more careful consideration, it may be cured of its defects and become a further means of placing construction in the service of economic and social advancement. The new appreciation of engineering construction as a primary part of the nation's industrial organism is a gain that may be expected to lead to still larger results in future.

Difficult Cofferdamming

FEW DAM SITES present more difficult problems in cofferdam construction than that encountered in the Bonneville development, and at none has more ingenuity been employed in developing the plan of attack. The river is swift and deep, the bottom is rough and irregular, and floodflows are so high that submergence of the cofferdam during the flood period of the first stage was recognized as an inevitable condition. The plan of construction adopted—floating in cribs and sinking them—is not new, but the refinements employed in designing the cribs, in preparing an accurate relief map of the bottom preparatory to building the cribs, and the use of models as an aid to the design studies, all are unusual elements of the undertaking that warrant careful study.

Safety Plugs for Dams

PREPARING for the unusual contingencies that may arise calls for the engineer's constant watchfulness and ingenuity. The safety-valve feature incorporated in a dam at Hornell, N. Y., to prevent its destruction by floodwaters, is a timely example of the results. In the construction of this dam a section of corewall on the land side of the spillway was omitted, so that the embankment could fail at this point under severe flood stress and thus enlarge the spillway channel before the main part of the dam could be overtopped. During the unprecedented July floods the water rose within a few inches of the predetermined safety blowout level, estimated to be a 500-year possibility, and design predictions were confirmed when sections of the riprap disintegrated and widened the channel to accommodate the unusual runoff. The main dam suffered no damage, and repairs to the blowout section were quickly made at little cost. In striking contrast was the destruction suffered by an older dam a short distance downstream, which was overtopped and washed out by the same flood. Since the wide extent of damage to engineering structures from the abnormal rains of the current year directs special attention to means of protecting against flood destruction, the successful safety-valve device tried at Hornell may find useful application in other dams where conditions limit spillway capacity and where the emergency discharge can be controlled by its use.

Lessons from Wreckage

NO MORE PROFITABLE FIELD OF STUDY has been available to the structural engineer than the crumbled masses and twisted members of some piece of his handiwork that met forces too great for its capacity. Out of piles of wreckage he has had to draw many warnings and precepts to guide the development of his art. Welding, one of his newer forms of expression, has so far given him little occasion to check his methods in this way in the light of disaster. Exceptions are a welded oil-storage tank that collapsed a year ago in Rhode Island and a penstock that failed soon after in West Virginia. Unfortunately, the owners of these structures have chosen to withhold their findings from the profession, so that the cause of the fatal accidents will never play a part in averting further loss of life. But if none of the other scores of welded structures have failed, which is cause for satisfaction, the completion this week of the deliberate wrecking of the 700-ft. towers of the Skyride in Chicago is an event worthy of study. By dropping these tall pieces of structural framework, violent punishment was administered to the 19 miles of shop welds used in fabricating the columns, struts and bracing members. Pending the results of complete studies, it may be noted that examination of the wreckage of the west tower, first to be felled, indicates that the welds withstood the impact forces well. There were instances of poor penetration, but they were not numerous or pronounced; on the other hand a great many of the welds were so strong that failure occurred in the body of the member rather than in the weld metal. The results cast no reflection on the art of structural welding as it has been developed to date, and leave a satisfactory feeling that present-day welds have real value under impact.

Practice and Originality

WHOEVER has observed the changes in his own field of engineering activity has undoubtedly been impressed with the strong trend toward uniformity of practice that exists at any given time, and the progressive shift in trend from epoch to epoch. The phenomenon is often so marked as to suggest the view that practice follows a prevailing style or fashion, and changes with change in the style. Even where this condition is less strikingly displayed it may be seen that the standards of one decade frequently undergo a thorough change in the following decade, and not infrequently return later on to the earlier standards.

Observers from across the water are in a peculiarly good position to notice this state of affairs. A like sequence of trends of practice is not unknown abroad, but it appears to manifest itself more strongly in the United States. A recent European commentator on sewage treatment, familiar with this country by frequent visits, speaks expressly of the greater tendency here than overseas to follow a prevailing trend or mode of engineering practice.

Does this drift toward uniformity of practice reflect a lack of originality? We think not. Rather, there are grounds for believing that it is closely related to progress, and that both in the aggregate and in the case of the individual project it testifies to a definite seeking for practical efficiency—in many instances of a higher type than could be realized if each new problem were approached as though it were the first of its kind to be encountered.

Development of technical skill and mechanical aids is undoubtedly a factor in causing practice to follow closely along the same line. Thus, the excellence achieved in certain forms of construction or equipment useful in the treatment of sewage is a definite and practically important reason for shaping practice in a particular way; the state of knowledge and evolution of constructional skill and apparatus applicable to concrete placing has a necessary bearing on forms of design in building, dam construction, and whatnot. The abilities and equipment of fabricating shop and field erector are important influences on steel bridge design.

Perhaps more important than these influences is the ever-present tendency toward standardization in each single department of engineering activity. This in turn is unmistakably an element of progress. It aims at the simplification of non-essentials and the elimination of arbitrary variation in order that the essentials may receive proper attention. And the fact that such standards do not (and should not) remain fixed but remain subject to revision even though this may mean return to a former standard at one time abandoned, will often create the appearance of a blind style trend in engineering practice, and appearance on closer study is seen to be utterly opposed to the facts.

Many other causes operate to bring about prevailing trends of practice in engineering work, but it is not our purpose here to do more than point out the existence of real and valuable reasons for a conformity to pattern that at times may appear like mere following of precedent rather than direct response to the requirements of the case. That such conformity to prevailing practice does not conflict with free exercise of originality and is not incompatible with thorough study of local factors has been proved by too many brilliant examples of engineering construction to permit of any misconception. The careful engineer at no time leaves out of sight the full range of expedients and solutions applicable to his problem, and never forgets the prime necessity of studying local conditions first of all before considering a solution. So long as this is done there can be no disadvantage in recognizing the full value of prevailing trends of practice.

Firedamp Leakage

SOMETHING NEW in connection with a danger long recognized is brought to light by the fatal explosion in the Chicago waterworks crib last week. At bottom the accident is only one more in a long series of fires and explosions caused by marsh gas contained in the clays of the Great Lakes area, but it differs from its predecessors by involving a finished structure.

Explosions or fires during construction in clay have occurred in both tunnel and caisson work, under water and on land. Because of its circumstances and the exceptional precautions that were put into force afterward, the explosion in the Cleveland water tunnel twenty years ago was perhaps most prominent; essentially, however, all accidents of the group showed same sequence of events—unnoticed and unexpected accumulation of firedamp seeping out of the clay, and chance ignition by a light or a spark when the gas mixture reached explosive proportions. Whether the gas flows in continuously at slow rate or whether larger volumes come in suddenly when a pocket or seam is opened by the excavation, the danger is equally serious, as the presence of the gas is not perceptible to the senses, and it may be in the air of the excavation for a long time without reaching the concentration necessary for combustion. Ventilation is therefore a first essential, and regular testing for the presence of firedamp is hardly less important.

The Chicago experience now brings the further warning that the gas danger may affect a finished structure. The firedamp entered the tunnel, which was partly empty, through joints and seams of the lining and waited only the favorable moment for an explosive mixture to find an igniting spark. Supplemented by this demonstration, the record of firedamp exposions in the clays of the Great Lakes area leaves no further question as to the need for care in connection with any work or structures in this material. The lessons of the past should make further accidents of the kind impossible.

CURRENT NEWS

PWA Applications May Be Revised After Deadline

Washington Correspondence

QUESTIONS propounded to Secretary Ickes on Aug. 29 threw some additional light on the deadliness of the deadline for relief works applications set by President Roosevelt. Insofar as PWA applications are concerned, a bona fide application received before the date set can subsequently be reopened for modification. This does not mean, however, that dummy applications omitting essential information can be filed as a subterfuge to evade the time limit. Asked if projects which required legislative action but otherwise complete, could be filed subject to subsequent ratification, the Secretary hedged. "Theoretically," he said, "we could not delay beyond the last meeting of the Advisory Committee on Allotments, set for Sept. 17, but we might wait a few hours longer if there seemed to be a reasonable expectation of prompt legislation." He pointed out, however, that in practically every such case the legislature concerned had already had ample opportunity to pass the necessary legislation.

Applications to be in on Sept. 7

Stimulated by the Sept. 12 deadline, local public bodies had, up to Aug. 29, filed with PWA 4,144 applications requesting $876,511,626 of federal money in addition to approximately $500,000,000 to be obtained from other sources.

Although the deadline for receiving applications has been set at Sept. 12, all state PWA directors have been instructed to have their applications in Washington not later than Sept. 7, so that they may be rechecked before submission to the Advisory Committee on Allotments.

Adminstrator Ickes has sent 386 telegrams to states, counties and cities applying for $49,168,359 in relief works funds for construction projects stating that these applications must be revised and calling their attention to the Sept. 17 deadline for final action by the Advisory Committee on Allotments. The applications had been rejected by WPA Administrator Hopkins because the cost per man was too high.

Stating that the immediate objective of the Works Progress Administration had been attained in Alabama and Wyoming, Administrator Hopkins has announced that direct federal relief in those states has come to an end. Nearly 33,000 needy persons have been put to work in the two states recently, it is reported.

In nine other states (Delaware, Mississippi, New Mexico, Oregon, Rhode Island, Texas, Vermont, Virginia, Wisconsin) September relief allotments are being withheld pending determination of the amount needed. New Hampshire and New Jersey have each been granted $100,000, indicating that the transfer of workers from relief will not become effective until October 1.

State Approves Revised Plan For San Gabriel Dam No. 1

Approval by the state of revised plans for the construction of San Gabriel Dam No. 1 has been announced by the Los Angeles County board of supervisors. The construction of the dam is a project of the Los Angeles, Calif., County Flood Control District. Notification of approval was received by the board from Edward Hyatt, state engineer. The construction cost of the redesigned dam will be $11,570,000. As originally designed the cost of the dam was estimated at $9,734,380. Contract for the structure was awarded to the West Slope Construction Co. on a bid of $8,600,527.

Construction work on the dam was stopped almost nine months ago. Revised plans calling for a larger structure and permitting the use of material for fill that was considered waste under the original specifications were approved July 26 by the board of supervisors and submitted to the state engineer. The work was to be resumed immediately following the ruling of the court. The flood control district has agreed to pay $13,033,532.26 for the completed work, of which $3,632,605.82 has been paid. This leaves an additional $9,400,926.54 to be paid the contractors out of $11,490,553.76 remaining from bonds voted for the project.

Work On Parker Dam To Be Resumed; TVA Amendment Signed

ORDERS to resume construction of Parker Dam on the Colorado River were issued on Aug. 31 by Secretary of the Interior Harold L. Ickes. Work on the dam has been at a standstill since April 30 when the United States Supreme Court decided that the consent of Congress must be obtained before the dam could be built. Work had been in progress for three months when the Supreme Court ruling was made. The Rivers and Harbors Bill, signed by the President, carries authorization for construction of Parker Dam across the Colorado River between Arizona and California, and also the construction of Grand Coulee Dam across the Columbia River in Washington.

Parker Dam, which is located 145 mi. downstream from Boulder Dam, is being built by the Bureau of Reclamation under contract with the Metropolitan Water District of Los Angeles at an estimated cost of $13,000,000. Its principal function is to provide a supply of water, which is to be pumped from the river into the Los Angeles aqueduct running about 220 mi. to

TWO DIVERSION TUNNELS at Parker Dam on the Colorado River were under construction when work was stopped last April as a result of the Supreme Court's decision that the work was not authorized by Congress. Six Companies, Inc., has the contract.

the Metropolitan Water District in Southern California.

Under the provisions of the contract about 80,000 horsepower is to be developed at Parker Dam, one-half of which has been allocated, without cost, to the State of Arizona, to be used in connection with irrigation development in the Gila Valley.

Construction work on Grand Coulee Dam in Washington has been in progress for the past eighteen months. The Supreme Court decision did not apply specifically to this structure. Its inclusion in the river and harbor act removes any objection that might be raised as to legal authorization for its construction.

TVA amendment signed

An act amending the original TVA Act, clarifying the powers of TVA and remedying certain defects that were evident after its two years of operation, has been signed by President Roosevelt.

The right to sell surplus power, which had been challenged by court actions on injunctions brought under the original TVA Act of 1933, is given definite sanction in the newly signed law. Norris, Wheeler and Pickwick Landing Dams are mentioned as being necessary to a unified Tennessee River system for navigation, flood control and power. TVA is authorized to use bond issues up to $50,000,000 for loans to cities and states enabling them to build or acquire distribution systems.

Power to construct additional dams in the Tennessee Valley in order to maintain a nine-foot channel is also granted as is the power of acquiring existing facilities for distribution of electric power. Regulation of power resale rate schedules is also granted.

Sea-Level Canal Across Florida Authorized by the President

President Roosevelt has approved an allotment of $5,000,000 to the Corps of Engineers for preliminary construction work on a sea-level ship canal across Florida. The canal will extend by way of the St. Johns River from its mouth to Palatka, thence, by way of the Oklawaha and Withlacoochee rivers, entering the Gulf near Port Inglis. This route will save ocean-going vessels from one to two and one-half days. Annual saving in transportation costs is estimated at $7,500,000.

Work will be started immediately. The allotment will be expended as follows: Clearing right-of-way, $500,000; housing, shops, storehouses and minor structures, $500,000; excavation in central areas, $3,500,000; bridge foundations, $500,000.

Contracts Awarded for Casper-Alcova Work

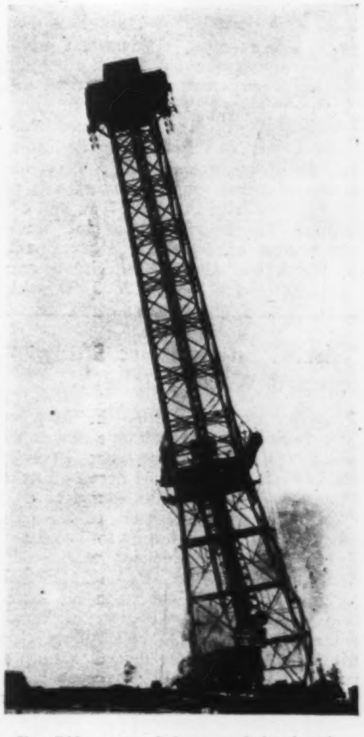
Contracts for the construction of two PWA dams in the Casper-Alcova project, Wyo., being carried out by the U. S. Bureau of Reclamation, were awarded on Aug. 29. The contracts awarded were for the Seminoe dam, including power plant, and the Alcova dam. Winston Bros. Co., Minneapolis, Minn., was awarded the contract for the construction of Seminoe dam and power plant on a bid of $2,194,007. The contract for the construction of Alcova dam was awarded to W. E. Callahan, Dallas, Tex., and Gunther & Shirley, Los Angeles, Calif., on a joint bid of $1,482,651.

Last of Large Structures Razed At Chicago World Fair

The east tower, 620 ft. high and weighing 1500 tons, of the Sky Ride on the Chicago Century of Progress grounds, was toppled on Aug. 31 by melting two of the four supporting legs with thermite. Dynamite was used to raze the west tower (*ENR*, June 20, p. 892).

Two cupolas, 10 ft. apart, each containing 750 lb. of thermite, proportioned to produce 5,500 deg. temperature, were built around each of the two north legs. The thermite was touched off by throwing an electric switch on the west side of the lagoon.

In falling, the tower appeared first to settle, then to fall apart at the 200-ft. platform level. As examined on the ground the top portion of the tower legs was essentially intact and straight, though all kinds of failure in shearing, bending and compression of both welded and riveted joints was observed. Early in the fall the back or south legs kicked backward, one breaking at a panel point 50 ft. up and another 25 ft. up. The tower was laid down with the top about 100 ft. out of line, being inclined to the east due to the dragging weight of the east back stay cables which had not been cut or loosened from the anchorages.



Sky Ride tower being razed by burning out a 10-ft. section of the north legs with 3,000 lb. of thermite. Front legs have dropped and the back legs are beginning to kick backward.

Demolition of the structures of the Century of Progress is being carried out by the Barker-Goldman-Lubin Co., for whom Frank A. Randall is consulting engineer and George Lindburg is general superintendent. J. L. McConnell is engineer for the Century of Progress and L. R. Lohr, general manager.

Floods Wash Out Dam in Colorado and Change River Channel in Nebraska

Floods caused by heavy rains in eastern Colorado on Aug. 28 burst through two small dams near Holly, swept away several farm houses and inundated the main highway as well as washing out sections of railroad track.

According to reports from M. C. Hinderlider, Colorado state engineer, and from D. W. Ormsbee of the state highway department, the Horse Creek flood control dam, an earth dike 40 ft. high, 2,300 ft. long, located about 10 mi. north of Holly, with a capacity of about 2,500 acre-ft., failed on Aug. 28, as a result of being overtopped by unprecedented runoff above the reservoir during a period of two hours. A major portion of the dam was carried away. The spillway at the west end of the dam, which is 225 ft. wide and 5 ft. deep, although of greater area than the natural channel of Horse Creek, was unable to accommodate the flood. There was practically no water in the reservoir and the outlet valve was open at the time of the flood.

Horse Creek dam was built by CWA and FERA forces, under state inspection, and was considered to have been constructed on sound engineering principles. There were no lives lost, as a result of ample warning, and property damage was limited to inundation of basements in Holly, rupture of the Santa Fe railroad tracks and the loss of one bridge. It is considered that the dam prevented a greater catastrophe by temporarily restraining the flood which would have reached greater proportions on the incidence of flood waters entering Horse Creek through a tributary below the dam.

At the time that the Horse Creek dam was wiped out, Holly was already under water to a depth of 6 in. to 1 ft. As a result of the flood the power plant at Holly failed and telephone and telegraph lines were put out of service.

Republican River channel altered

From 2 to 4 in. of rain on the watershed from Oxford, Nebr., and extending about 100 miles west, sent the Republican River in Nebraska out of its banks and threatened to reach the flood proportions that occurred in the early part of June. McCook, which was particularly hard hit in the last flood, again suffered heavy damages as the flood waters cut off all but one traffic outlet.

Reports from V. F. Osborn, project engineer of the state department of roads and irrigation, Oxford, Nebra., indicate that 1.60 in. of rain fell there on Aug. 24 and 25. Muddy Creek and Medicine Creek were the principal sources of flood water

(Continued on p. 343)

Injunctions Forbid PWA Funds For Municipal Power Plants

A preliminary injunction forbidding PWA to lend money to cities for power plant construction was granted on Aug. 28 by the District of Columbia Supreme Court. The action was brought by the Kansas Utilities Co. and the Texas Utilities Co. and involved, specifically, PWA loans to the cities of Burlington, Kan., and Plainview, Tex.

Pending action by the U. S. Supreme Court on a test case, PWA officials indicated that they would continue to receive and pass upon other applications for municipal power plant allotments.

Illinois city to appeal injunction

According to information issued by the Mayor, the city of Centralia, Ill., is expected to appeal from the recent decision of the United States District Court for the Eastern District of Illinois at Danville, Ill., in granting a permanent injunction restraining the city from proceeding with the construction of a municipal light and power plant. The litigation against the municipal project was instituted by the Illinois Power & Light Co. The injunction, which was granted on Aug. 1, held that the proposal for the project was not in compliance with the state laws and constitution. The court declined to pass on the question of federal constitutionality. The PWA had allotted the city a grant of $117,000 and a loan of $300,000, which was to be supported by $300,000 in bonds voted by the city.

Only seven of the 31 PWA projects for municipal power plants have escaped without challenge. Court decisions to date have had little effect; the U. S. Supreme Court is the only arbiter whose decision will be accepted as final. There is a case to be tried this fall challenging a power plant in the city of California, Mo., which was completed Dec. 31, 1934, and is now in operation.

There is no "yardstick" idea connected with these municipal plants financed through PWA. Theoretically, it makes no difference to that body whether an application for funds covers paving, sewers or a power plant; if the requirements are satisfied the loan goes through.

Contracts Allowed to Provide For Future Wage Rules

President Roosevelt has signed a Congressional resolution permitting government agencies to award contracts on which bidders have agreed to adhere to NRA codes, provided the low bidder will agree to modify such contracts to conform to future minimum wage, maximum hour and minimum age restrictions which may be enacted by Congress. Provision is made for adjustment of compensation in such cases.

The purpose of this legislation is to avoid the necessity of readvertising a number of contracts upon which bids were prepared before the Supreme Court's nullification of NRA. The Comptroller-General has ruled that, without enabling legislation, such bids could not be accepted. The President's signing of the recent resolution restored these contracts to full legal status.

Montreal Engineers to Study Proposed Water Intake

Proposed expenditure by the Montreal Water Board of $1,200,000 on a new intake structure in the St. Lawrence River at Ville LaSalle has advanced a stage further by the appointment by the executive committee of the City Council of an independent board of engineers to investigate and report on the proposed work. The board consists of Prof. R. de L. French, A. Plamondon, J. M. Robertson, E. Gohier, and J. E. Blanchard, Director of Works for the city with C. E. DesBaillets, chief engineer to the water board acting as secretary. As a result of recent heated controversy in civic circles, it has been shown that the city authorities have been repeatedly warned since 1921 by Water Board officials that consumption was exceeding the intake capacity, that the untreated shore water being obtained from the existing intake was undesirable for domestic purposes, with the result that a new intake was an economic necessity, considering only the treatment of present supplies.

Sewer Work to Start

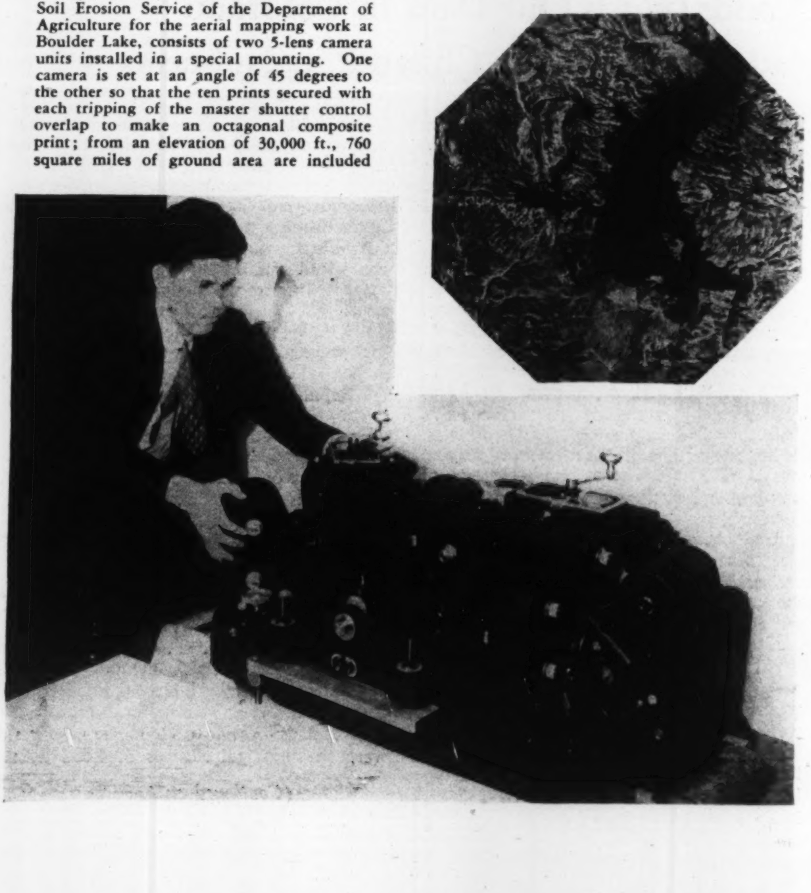
Expenditure of $200,000 out of a projected amount of $657,000, on sewer works in Montreal has now been approved by the Quebec Municipal Commission, and work is due to start shortly. The work will be undertaken both by contract and by daylabour, the latter because it has been impossible to locate exactly certain of the very old sewers which are to be disconnected, so that contract plans cannot be prepared.

TEN-LENS CAMERA AND ONE OF ITS SHOTS OF BOULDER LAKE

For the purpose of defining contours of the bed of the reservoir at Boulder Dam, aerial photographs are being taken each time the water in the reservoir rises 20 ft. These 20-ft. contours will furnish a guide for a topographic map with 5-ft. contours to be used in studying the progressive silt accumulations in the reservoir. In this work the world's largest aerial camera is taking the largest photographs ever made.

The accompanying views show the camera and one of the pictures. The latter, taken from a height of 20,000 ft., shows 200 square mi. of Nevada and Arizona and the lake at the highest level it is likely to attain this year. Boulder Dam, in the lower right hand corner, is barely visible from this great height of nearly four mi. The camera developed by Fairchild Aerial Surveys, which has the contract with the Soil Erosion Service of the Department of Agriculture for the aerial mapping work at Boulder Lake, consists of two 5-lens camera units installed in a special mounting. One camera is set at an angle of 45 degrees to the other so that the ten prints secured with each tripping of the master shutter control overlap to make an octagonal composite print; from an elevation of 30,000 ft., 760 square miles of ground area are included in one composite photo. By taking in such a wide expanse of territory the camera becomes particularly useful in securing ground control points for accurate mapping. It is being used for this purpose on another contract covering the rugged terrain of the Rio Grande Valley in Central New Mexico.

The assembled camera unit will film roll weighs 275 lb. The ten rolls of film needed for a single loading weigh 70 lb., and include 1,200 ft. of film, sufficient for 2,000 individual photographs or 200 composites of ten exposures each. Each set of ten prints measures 32x32 in. The ten shutters are operated electrically at the same instant. Should any of the shutters fail to function properly, a small neon light flashes a warning signal.



Washington Highlights

By Paul Wooton

In one of his last communications to Congress before adjournment, President Roosevelt suggested that studies during the recess by the Senate Finance Committee and the Ways and Means Committee of the House be followed by public hearings for "the formulation of proposed legislation for preserving permanently to the nation such social and economic advantages as were gained through previous emergency enactments." This is a further indication of the President's determination to press for NRA legislation that will not fly in the face of the Supreme Court's decision.

Restoration of the NRA principle is regarded essential in Administration quarters if the shift of the textile industry away from New England is to be checked. The recent report of the cabinet committee is being cited to show that the migration of that industry to the South, which was proceeding rapidly prior to NRA, was halted by the operation of that plan.

Establishment of five regional offices has been announced by the Federal Power Commission. Each office will have an operating staff. Cooperation wth state commissions will be largely through the regional organizations.

Washington, September 3, 1935.

Hotchkiss Begins Duties At Rensselaer

Dr. William Otis Hotchkiss began his duties as president of Rensselaer Polytechnic Institute on Sept. 3. Dr. Hotchkiss had been president of the Michigan College of Mining and Technology for ten years preceding his election to succeed the late president of Rensselaer Polytechnic Institute, Dr. Palmer Chamberlain Ricketts, who died last December.

As state geologist in Wisconsin, Dr. Hotchkiss is credited with originating the state's highway program and in carrying out the program by serving on the highway commission from 1911 until 1925, when he became president of the mining college. Dr. Hotchkiss is a graduate of the University of Wisconsin and holds degrees of civil engineer and doctor of philosophy from that university.

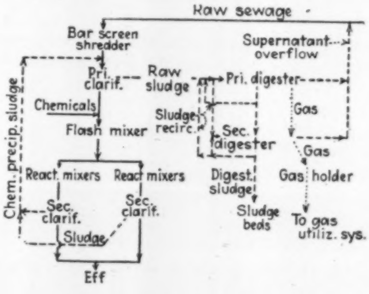
Plan for Highway Sidewalks Advanced in Massachusetts

Plans for constructing sidewalks and curbings along a total of 1,000 mi. of state highways in Massachusetts at an estimated cost of $8,000,000 have been agreed upon by Arthur G. Rotch, ERA and WPA state administrator, and William F. Callahan, commissioner of public works. Half the cost would be borne by the state and half by the federal government. Mr. Rotch anticipates that the plan will be approved by the Washington authorities, and the work will be done in two phases, the federal government doing the grading, graveling and drainage work and the state the sidewalk construction and placing of curbs. Sidewalks will be built on one side of the highway only and will be from 4 to 6 ft. wide. Most of the construction will be of bituminous concrete.

Colorado Springs to Build Chemical Sewage Plant

Following the example of the St. Paul-Minneapolis Sanitary District and of New York City at its Coney Island plant in providing chemical treatment of sewage for part of the year only, Colorado Springs, Colo., has embarked on a similar program for a 10 m.g.d. plant. Treatment is to be provided during the irrigation season.

Bids for equipment are to be opened Sept. 7. When these bids are in and equipment has been chosen the designs for the plant will be made. At present only a flow sheet similar to that submitted with the PWA application is available. PWA allotment has been made and bonds sold.



Flow sheet of chemical precipitation sewage treatment plant for Colorado Springs which has been given to bidders on the mechanical equipment. Only after the bids are in will the actual plant designs be made.

In the specifications many equipment alternates are described "to insure competitive bidding" but alternates not covered in the specifications will be given no consideration. Apparatus submitted must have been in successful operation 18 months or have been installed or be in the course of installation in five representative American cities in sewage treatment plants.

Colorado and Nebraska Flood

(Continued from p. 341)

at this time. The Aug. 25 flood height at Oxford was within 7 ft. of the June 1 height. Intensity of the discharge was greatly increased by the new channels formed during the past floods. The report further states that the damage during this flood was caused by the shifting of the channels and floating debris. The entire valley for miles up and down stream is a potential channel and the stream in shifting has left bridges over old channels and has caused delay in rebuilding highways and public utilities, besides destroying hundreds of acres of farm land.

The new banks are too low to be successfully protected with the limited funds, material and equipment on hand, and the procedure now is to put in temporary structures and wait to determine where the river will finally locate its channel.

One-half mile of temporary grade and a 100 ft. FERA temporary trestle were lost at Oxford, as was 120 ft. of a new trestle in course of construction which was rendered useless by the channel having been altered so as to run parallel to instead of at right angles to the bridge site.

SOCIETY CALENDAR

AMERICAN WELDING SOCIETY, fall meeting, Chicago, Ill., September 30 to October 4.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, Milwaukee, Wis., October 7-10.

NATIONAL SAFETY COUNCIL, annual congress, Louisville, Ky., October 14-18.

PUBLIC WORKS ENGINEERING CONFERENCE, joint meeting of the American Society of Municipal Engineers and International Association of Public Works Officials, Cincinnati, Ohio, October 14-16.

INSTITUTE OF TRAFFIC ENGINEERS, Sixth annual meeting at Louisville, Ky., October 15-16.

AMERICAN SOCIETY OF CIVIL ENGINEERS, fall meeting, Birmingham, Ala., October 16-18.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, White Sulphur Springs, W. Va., October 16-18.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual meeting, Columbus, Onio, October 23-25.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual convention, Miami, Fla., December 9-12.

ROCKY MOUNTAIN SECTION, American Water Works Association, annual meeting, Denver, Colo., September 16-18.

NEW ENGLAND WATER WORKS ASSOCIATION, fall convention, Providence, R. I., September 17-20.

NEW ENGLAND AND NEW YORK STATE SEWAGE WORKS ASSOCIATIONS, joint meeting at Schenectady, N. Y., October 4-5.

EXAMINATIONS for registration of engineers and land surveyors will be held in the near future as follows:

LOUISIANA—By Louisiana State Board of Engineering Examiners at the offices of the Board, New Orleans, Sept. 9-10.

Obituary

Robert E. Norvell, 65, Hugo, Colo., superintendent of maintenance for the state highway department, died August 24, after a long illness.

William Bailey, contractor's superintendent, died Aug. 28. For six years, Mr. Bailey superintended the building of tunnels and also several sections of the H.&M. R.R. North River and the Pennsylvania R.R. East River tunnels. In Mexico he had charge of driving tunnels for carrying water to Mexico City, for the Mexican Light and Power Co. While in the employ of the Degnon Construction Co., for four years, he directed the underpinning of the tracks and station at the Atlantic Avenue Station, Brooklyn, for the B. M. T. Subway. He worked for the Arundel Construction Co., superintending open-cut work on the new Pennsylvania Railroad Union Tunnel in Baltimore, Md. While engaged there he was taken ill.

Dr. Curtiss F. Marbut, for many years Chief of the Soil Survey of the United States Department of Agriculture, died in Harbin, Manchuria, on August 25. Doctor Marbut had passed the age of retirement, but had been retained in order to continue research on the classification of soils. At the time of his death, Doctor Marbut was on his way to make a study of the soils of China.

Henry Adams, British consulting engineer, died in London late in August, at the age of 89. He was educated at King's College, London, and the City of London College. He took up private engineering practice in 1877, and after 1909 was in partnership with his son. The chief kind of works carried out by him were coal stores, wharves, jetties, hydraulic machinery, workshops, offices, water supply and sewerage, and structural steelwork and reinforced concrete. He was a chief examiner in engineering for the board of education from 1905 to 1910. He was also superintending examiner for the Society of Engineers.

George E. Fritcher, retired civil engineer, died on Sept. 1 at his home in Pasadena, Calif., at the age of 83. He had at various times maintained offices in Chattanooga, Tenn., and New York City.

He was born in Sharon Springs, N. Y., and received his engineering training at Rensselaer Polytechnic Institute. He began work on a governmental survey of the Missouri River from Fort Leavenworth to the mouth of the river. Mr. Fritcher later served for three years as chief engineer for the United Fruit Co., and it was while holding this position that he engaged in most of his foreign projects, which were concerned with railroad building in South America. In 1890 he established engineering offices in New York City, and later moved to Chattanooga to become an instructor in engineering. Mr. Fritcher retired from active work three years ago and moved to Pasadena.

William M. Duane, PWA resident engineer in Cincinnati, Ohio, died there on Aug. 22, at the age of 68. Mr. Duane was graduated from the Massachusetts Institute of Technology in 1889 and became assistant engineer on the Cincinnati and Muskingum Valley Ry. He later took a similar position on the Indianapolis division of the Pennsylvania R R. and in 1891 became assistant chief engineer of the Peoria & Eastern Ry., but resigned after a few months to become engineer of maintenance of way for the Cleveland, Cincinnati, Chicago & St. Louis R. R. He later became superintendent of construction for the C.C.C.&St.L.R.R. and in 1907 was made chief engineer, which position he held for five years. He later became a member of the firm of Duane & Carlen, railroad contracting engineers.

Harry E. Stitt, who had been chief engineer of The Austin Co., engineers and builders, until recently, when he became consulting engineer, died in Cleveland, Ohio, Aug. 23, at the age of 58.

Mr. Stitt was graduated from Pennsylvania State College in civil engineering in 1903 and went with The Austin Co. as chief engineer in 1914.

CONSTRUCTION STATISTICS OF THE WEEK

High federal awards raise this week's engineering construction volume total above last week's in spite of the fact that this is a short week due to the Labor Day closing. The total for the week, $27,362,000 compares with $26,113,000 last week and is made up of private construction, $2,831,000, and public awards, $24,531,000 of which federal is $14,192,000 and state and municipal, $10,585,000. The corresponding values a year ago were: total, $19,572,000; private, $3,031,000; public, $16,541,000; federal, $3,504,000; state and municipal, $13,037,000.

The heavy awards this week were in the earthwork, irrigation, waterways classification which totalled $10,837,000. Highways, at $4,503,000, were higher than last week, while industrial, commercial and public buildings ran slightly below last week's volume. Sewerage awards were higher, but waterworks and bridges were slightly down.

The larger awards for the week include hospital, Ft. Sam Houston, Tex., $1,347,000; highway awards by West Virginia, $542,000 and by Georgia, $1,582,000; sewers, Louisville, Ky., $433,000; Cont. 109, Divs. C. & G. also mechanical equipment, Minneapolis-St. Paul Sanitary District, Minn., $680,000; Dam No. 8, Mississippi River, Genoa, Wis., $2,047,000; Seminoe Dam and power plant, Casper-Alcova project, Casper, Wyo., $2,194,000; Alcova Dam, Casper-Alcova project, Casper, Wyo., $1,483,000; Dam 18, Mississippi River, Burlington, Ia., $2,515,000; Dam 11, Mississippi River, Dubuque, Ia., $2,359,000; improvements, Kelly Field, San Antonio, Tex., $524,000; and recreation center, day labor, relief rolls, Buffalo, N. Y., $900,000.

New capital for the week includes state and municipal bond sales, $3,077,000 and a transfer by sale through RFC of $13,198,000 from PWA financing to private financing. Federal PWA allotments for the week total $4,463,000 and the 25 per cent of WPA allotments included in the new capital for construction figures add $26,086,000 to the new capital total from federal funds and $4,095,000 from local funds.

CONTRACTS

(Thousands of Dollars)

	Weekly Sept. 1934	Average Prev.4 Weeks	Week Sept.5 1935
Federal Government	$4,256	$14,862	$14,192
State and municipal	15,021	13,002	10,339
Total public	$19,277	$27,864	$24,531
Total private	4,333	6,180	2,831
Week's total	$23,610	$34,044	$27,362

Cumulative to date:
1934...$959,661,000 1935...$928,735,000

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1935	Week Sept.5	Cumulative
State and municipal...	$16,275	$279,742
PWA allotments, S&M.	—13,198*	308,037
RFC loans, S&M.......	.....	36,084
Corporate issues	633	21,153
PWA private	.....	8,629
Local contrib. to WPA.	4,095	8,338
Total non-federal ...	$7,805	$661,983
PWA federal	30,549	874,163
Total new capital....	$38,354	$1,536,146

Cumulative to date:
1934..$1,091,822,000 1935..$1,536,146,000

*PWA issues sold for private investment.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for federal construction, deficiency act funds. An arbitrary percentage (25%) of the WPA allotments and local contributions to WPA work is included to allow for the capital additions through the Works Progress Administration division of the new program.

INDEX NUMBER

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Sept., 1934	200.58	96.41	Aug., 1934	94	41
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1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
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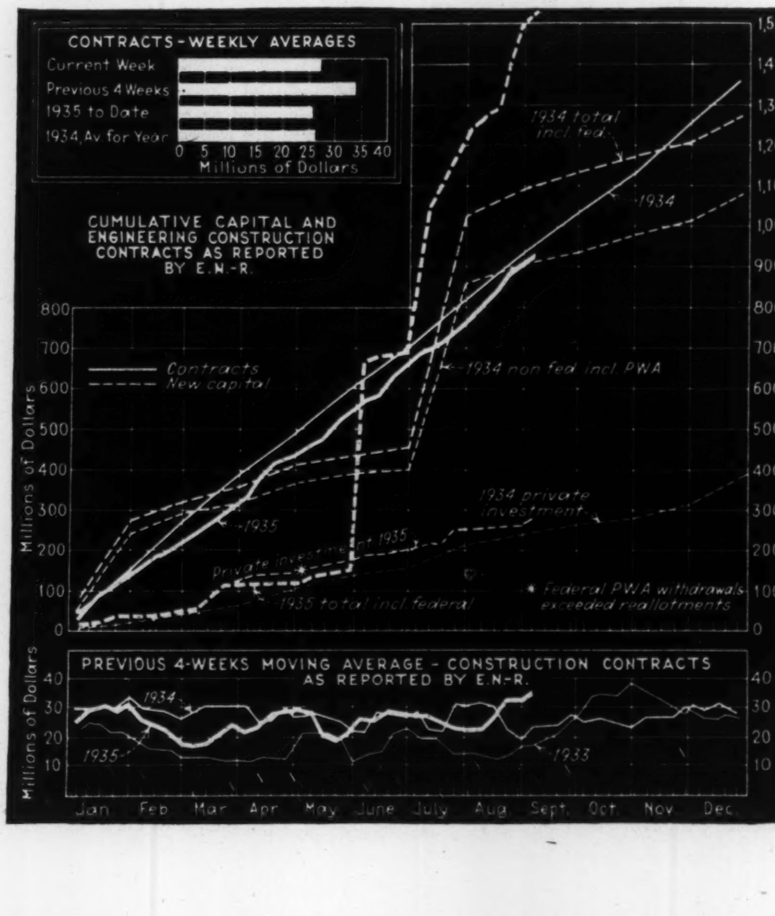


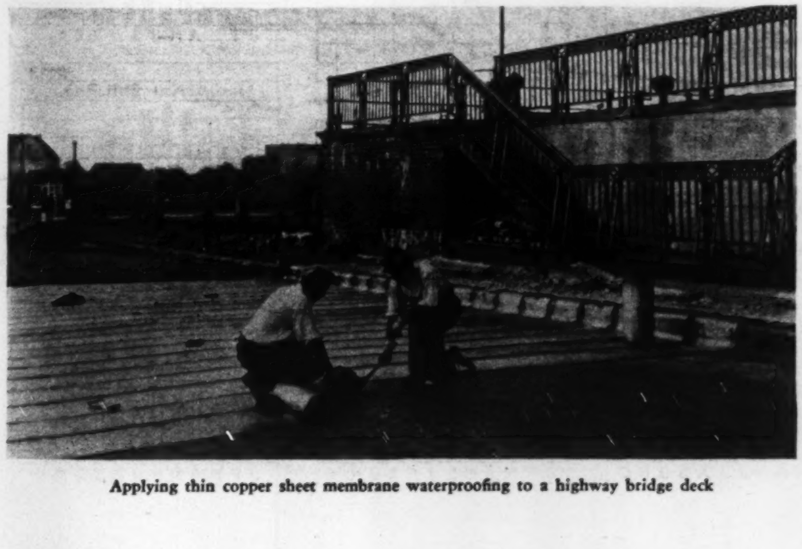
Construction Equipment and Materials

Thin Copper Sheets Used For Waterproofing

In Germany rather extensive use is made of thin copper sheets for waterproofing engineering structures. Pasted down with a special bituminous mixtures, the copper sheets are claimed to provide a permanent, watertight and economical method of waterproofing.

Although copper sheets 0.04 in. and more in thickness have been used commonly as water-stops in joints, the use of thin sheet membranes for waterproofing of entire constructions is relatively new. Such an insulation, known as HEKU and patented in the United States, is now being introduced into this country by the American Lurgi Corp., 80 Broad Street, New York City. It is stated that since this type of insulation was introduced in Germany about three years ago, over a million square feet has been used, of which 700,000 sq.ft. was applied in 1934 on about 200 structures, including bridges, tunnels and buildings and for both foundations and roofs. On the new bridge over the Elbe River at Meissen, 7,000 kg. of copper sheets 0.2 mm. thick were used, to cover a deck area of 3,500 sq. meters. The cost, including the bitumen and the installation, was about $2.40 per sq. meter.

The copper sheet insulation is applied as follows: The polished cement surface of the structure is first covered with a bitumen applied cold. Upon this prepared surface is applied a hot pulpy bituminous material in which the insulating strips of copper, 0.004 in. or 0.008 in. thick, are rolled out flat, pasting them down. The edges of adjacent strips are overlapped several inches, and an additional application of the hot bitumen seals the joints. During the pasting, the metal sheets become hot, expand and then contract when they cool off; this action is claimed to promote an improved bond. After the metal strips have all been laid in place, they are sprayed with a hot liquid asphalt as a final sealing coat.



Applying thin copper sheet membrane waterproofing to a highway bridge deck

All-Wheel-Drive Ford Truck Marmon-Herrington Equipped

By installing a specially designed front driving axle and propeller shaft and a two-speed auxiliary transmission on a Ford V-8 truck, the Marmon-Herrington Co., Indianapolis, Ind., has developed a light weight all-wheel-drive truck.

Marmon-Herrington Equipped Ford V-8

Production has been started on four models: two four-wheel-drive units, and two six-wheel-drive units. The four-wheel-drives have a gross capacity of 13,200 lb. and the six-wheel-drives a gross capacity of 22,000 lb. Including the all-wheel-drive Ford V-8 trucks, the complete line now includes 28 models ranging in gross capacity from 8,400 to more than 52,000 lb., some of which are equipped with diesel engines.

The two-speed auxiliary transmission gives a total of 10 speeds forward and 2 reverse. The front drive axles are full floating, single reduction.

Manitowoc Power Shovel Has New Boom Design

The new Manitowoc speed shovel, manufactured by the Manitowoc Engineering Works, Manitowoc, Wis., embodies new features in design and operation. Care has been taken to provide for greater accessibility and perfect lubrication; all gears are spline fitted to shafts, and all working parts of the machine are fully enclosed and operate in oil circulated under pressure. The only exposed working gear is the large ring gear and swing pinion.

These shovels, obtainable in 1 to 2 cu.yd. capacity, may be had equipped with either chain power take-off or helical gear drive. Simplicity of design in the working parts allows ample room in the cab. A 90 gal. fuel tank is built as part of the rear cab plate, and provides a work bench for the operator as well as allowing ample fuel capacity to operate the machine for several days. Motor controls are extended to the operator's station allowing for quick control of the power plant.

Tubular type shovel and crane booms are a new development and provide for greater strength with less front and weight. The dipper sticks are mounted on the underside of the shaft sprockets, taking advantage of the normal upward pressure of the dipper sticks when operating under load, and eliminating the conventional saddle blocks.

New Publications

SAUERMAN LONG RANGE MACHINES, Sauerman Bros., Inc., Chicago, Ill., 8½x11, 56 pages. Scrapers and cableways.

KING-CLIP GATE VALVES, The Lunkenheimer Co., Cincinnati, Ohio. 8½x11, 8 pages. Circular No. 504.

HARD-FACING AND OVERLAYING. Lincoln Electric Co., Cleveland, Ohio, 8½x11, 14 pages. Building up metallic working surfaces by welding.

REPUBLIC WIRE NAILS, Republic Steel Corp., Chicago, Ill. 6x9, 40 pages. Form Adv. 226-B.

BETTER SCREW ANCHORAGE, The Rawlplug Co., Inc., New York City, 3x5, 8 pages. Bulletin U-100.

SERIES AP WICO MAGNETOS, Wico Electric Co., Springfield, Mass. 8½x11 pages.

PNEUMATIC TIRES FOR WHEELBARROWS AND INDUSTRIAL TRUCKS. B. F. Goodrich Co., Akron, Ohio. 8½x11, 4 pages.

COMBINATION MERCURY VAPOR AND MAZDA LAMP REFLECTORS. The Miller Co., Meriden, Conn. 8½x11, 10 pages. Price lists.

LINK-BELT PORTABLE CONVEYORS, Link-Belt Co., Chicago. 8½x11, 4 pages. Folder No. 1414.

NEW DEPARTURE BALL BEARINGS, 10TH EDITION, New Departure Mfg. Co., Bristol, Conn. 5½x8½, 144 pages. Dimensions, load data, and list prices.

Business Notes

THE CHAIN BELT CO., Milwaukee, Wis., has appointed the Burford-Toothaker Tractor Co., Montgomery, Ala., as the exclusive distributor of REX construction equipment in that territory.

CHARLES C. KELLOGG & SONS CO., Utica, N. Y., has been awarded the franchise to represent the Air Conditioning Department of the General Electric Co., in that territory.

CHADWICK MACHINERY CO., Milwaukee, Wis., has been appointed distributor of Link-Belt crawler shovels and other equipment. The Chadwick Company has shop facilities for handling any type of repair job.

THE FOUR WHEEL DRIVE AUTO CO., Clintonville, Wis., celebrated its twenty-fifth anniversary with Walter A. Olen, D. J. Rohrer and Frank Gause, continuing their positions as president, treasurer and secretary, respectively, which offices they have held continuously since the inception of the company in 1910.

THE ELECTRIC STORAGE BATTERY CO., maker of Exide Batteries, has appointed Harry W. Beedle as manager of the Boston branch.

THE AMERICAN LUMBER & TREATING CO., has opened offices at 116 New Montgomery Street, San Francisco, Calif. in charge of Mr. Chas. R. Wilson who was formerly with the McCormick Lumber Co.

AIR REDUCTION SALES CO., moved their Portland, Ore., store to larger quarters at 13 N.W. 4th Ave.

BUSINESS SIDE OF CONSTRUCTION

August Awards Highest for Year Increase in Public Construction

A SHARP increase in federal awards pushed the August total for engineering construction up to $158,057,000 for a 5-week month. Of this total $32,196,000 is for private construction and $125,861,000 for public of which $63,005,000 represents federal awards. The corresponding figures last year were: total, $109,115,000; private, $20,215,000; public, $88,910,000; federal, $32,066,000. Private awards continue higher than a year ago with August registering a 59 per cent gain over last year. The August figures represent a gain over last month in all classifications.

Geographically, Middle Atlantic takes a wide lead both in volume and in gain over last year. All territories show gains over last month, while all but West of Mississippi show gains over last August. Middle West and New England show only slight gains over this month last year, while Middle Atlantic, South and Far West show broad gains.

Highway awards started to pick up and for the first time this year are heavier than during the corresponding month last year. Public building awards are heavier than last month and last year as are industrial buildings, while commercial buildings are lower this month than last, but ahead of last August. Industrial buildings for 8 months are 68 per cent ahead of last year.

New Capital

New capital reached the $1,500,000,000 mark compared with $1,100,000,000 [illegible] ago. Heavy federal allotments [illegible] course, responsible for this substantial increase, although private investment [illegible] capital is running 15 per cent higher [illegible] last year. Private investment capital [illegible] counts for $284,000,000 of this year's [illegible]lative total.

Materials

Cement—Cement shipments in July totalled 7,813,000 bbl., a decrease of 1.1 per cent compared with July, 1934. Cement stocks at mills were 6.6 per cent higher than a year ago as reported by the Bureau of Mines. Shipments for the first [illegible] months of this year were 7.7 per cent behind a year ago. The ratio of production to capacity was 35.3 for July, 1935 compared with 35.7 for July, 1934 and [illegible] for June, 1935.

Steel—The operating rate of steel companies having 98.7 per cent of the steel capacity of the industry averaged 47.7 during August compared with 40.4 during th[illegible]

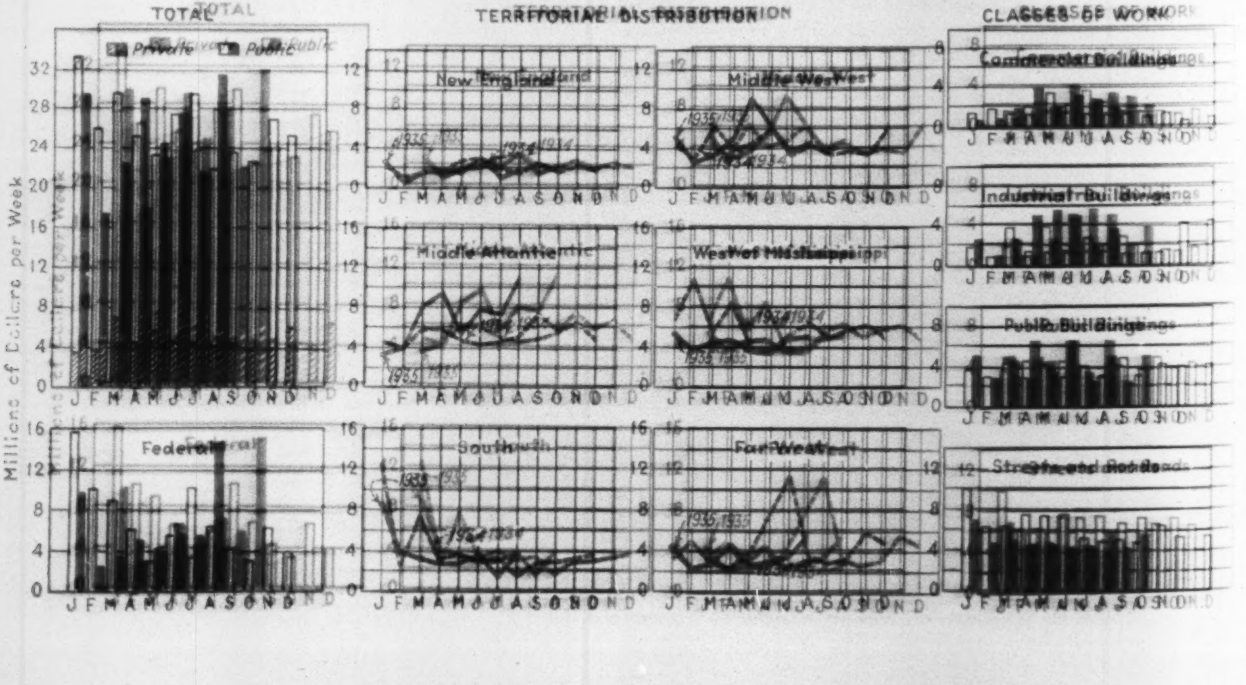
ENGINEERING CONSTRUCTION CONTRACTS REPORTED IN AUGUST, 1935

Five Weeks—Thousands of Dollars—(000 Omitted)

	New England	Middle Atlantic	South	Middle West	West of Mississippi	Far West	United States August 1935	United States Eight Months 1935	United States Eight Months 1934	Canada
Public Works										
Waterworks	116	741	618	227	788	6,684	9,174	46,456	48,411	217
Sewerage	495	1,554	200	1,432	274	119	4,074	44,154	38,823	[illegible]
Bridges, public	113	2,281	151	705	260	269	3,779	43,536	62,411	[illegible]
Earthwork and waterways	3,537	16,559	8,942	5,565	8,275	6,400	49,278	155,722	211,255	9,343
Streets and roads	1,595	2,725	5,565	5,623	7,787	5,514	28,809	173,558	236,857	3,847
Buildings, public	611	14,207	1,510	879	4,352	3,827	25,386	157,923	131,705	2,459
Unclassified, public	154	3,214	422	919	232	420	5,361	37,359	44,712	[illegible]
Total public	6,621	41,281	17,408	15,350	21,968	23,233	125,861	658,708	774,187	16,496
Federal gov't (included in above classifications)	3,881	24,260	10,280	6,180	9,807	8,597	63,005	219,311	283,429	
Private										
Bridges, private	....	909	....	264	25	....	1,198	9,416	7,839	
Buildings, industrial	720	7,002	982	5,293	2,160	1,554	17,711	116,755	69,503	[illegible]
Buildings, commercial	1,050	5,537	79	1,014	310	1,228	9,218	71,630	60,412	981
Unclassified, private	804	461	420	651	739	994	4,069	45,692	28,147	163
Total private	2,574	13,909	1,481	7,222	3,234	3,776	32,196	243,493	165,902	1,786
August, 1935 (5 weeks)	**9,195**	**55,190**	**18,889**	**22,572**	**25,202**	**27,009**	**158,057**	....	....	18,282
July, 1935 (4 weeks)	3,921	29,918	4,999	14,437	20,900	12,698	86,873	....	....	5,432
August, 1934 (5 weeks)	7,400	25,003	7,121	21,288	31,712	16,591	109,115	....	....	5,309
Eight months, 1935	59,491	247,017	133,874	174,914	158,180	128,725	....	902,201	....	57,447
Eight months, 1934	67,176	170,206	161,429	157,864	218,322	165,092	....	....	940,089	29,644

ENGINEERING CONSTRUCTION REPORTED BY E.N.-R.—WEEKLY AVERAGES

□ —1934 ■ —1935



preceding month and 22.1 for the corresponding period a year ago according to reports of the American Iron and Steel Institute.

The American Institute of Steel Construction reported for June that shops reporting to them, representing 78 per cent of the industry operated at 38.3 per cent of capacity. June bookings were 60 per cent larger than the average monthly bookings for the first 5 months of this year but about equalled bookings for the same month in 1934. June shipments were approximately 14 per cent less than in 1934. The backlog of the industry was approximately 34 per cent less than the previous year.

During July, production of fabricated steel was 38.5 per cent of normal, according to reports from shops representing 68 per cent of the industry. July shipments were 40.5 per cent of normal compared with an average of 36.6 for the 7 months.

Lumber—Lumber production jumped during the week ended July 27 to the highest total in 16 months while shipments and orders were heaviest since April of this year. Total production of reporting mills was 39 per cent above corresponding week of last year with shipments 46 per cent and new business 13 per cent heavier, according to report of the National Lumber Manufacturers Association.

Lumber consumption for the first half of 1935 was estimated at 8,002,000,000 compared with 7,884,000,000 in 1934. It is expected that total production will be about the same as last year.

Following the record production in the last week of July, each week of the third quarter has seen reported lumber production above the preceding week and with one exception shipments have likewise increased. Shipments during the week ended August 17 were highest of any week of the year except two in April. Production is running about 222,000,000 ft., shipments, 200,000,000 ft., and new business 195,000,000 ft.

Building Permits

The July record of building permits reported by Dun & Bradstreet, Inc., for 215 cities in the United States increased to $54,158,182, a 2.8 per cent gain over June and a 60 per cent increase over July a year ago. This is the largest total reported since November, 1931. New York City's building permit total for July was $11,213,613, 14.2 per cent below June but 41.8 per cent ahead of July a year ago. Permits for July in the 214 cities outside of New York totalled $42,944,569, an increase of 8.4 over last month and 65.2 per cent over last year. By territories the change from last year is: New England, down 42.7 per cent; Middle Atlantic, up 33.1 per cent; South Atlantic, up 102.8; East Central, up 93.5; South Central, up 102.7; West Central, up 199.1; Mountain, up 85.4; and Pacific, up 125.5 per cent.

The Bureau of Labor Statistics, U. S. Department of Labor reports for July a 3 per cent gain in number of permits, but a 1 per cent decrease in indicated expenditures, based on reports from 760 identical cities having a population of 10,000 or more. Comparison with July, 1934, by type of building, indicates an increase this July, based on estimated cost of 214.9 per cent in new residential, 18.4 per cent in new non-residential, and an increase of 35.9 per cent in additions, alterations, repairs.

Prices

Price changes this month are scattered and local. In New York City cement dropped 10c. per barrel, apparently due to the influence of Belgian imports. Sand, gravel and stone dropped 20c. a yard in Los Angeles. Structural clay building tile took a big drop in Cincinnati and a smaller drop in St. Louis. Asphalt prices are lower in Cleveland, and up in Dallas. Roofing is lower in Boston but up in Dallas. Lumber prices in Boston are $1 per M lower on the 1-in. material and $2 per M lower on the 2-in. material, both Southern Pine and Douglas Fir. At Denver and Kansas City, however, Douglas Fir is higher, from $35 to $40 on the 1-in. at Denver and $41 to $44 for 2 x 4-in. and at Kansas City from $24.50 to $27 for 1 x 6-in. and $26.75 to $28 for 2 x 4-in. At both New Orleans and St. Louis, Southern Pine prices are down, $1 per M. at the latter center and various amounts according to size at New Orleans.

Labor

Employment in the production of six major construction materials as reported by the Federal Employment Stabilization Office is 45.5 for July, 1935 compared with 44.6 in June and 43.0 in July, 1934.

According to the Division of Statistics and Information, New York State Department of Labor, lumber and building materials and hardware stores increased their forces in June for the third consecutive month. An increase of 1.6 per cent in employment was reported in June.

Labor controversies during the month centered principally around the rates paid on Works Progress Administration jobs. The New York strike ended when the "strike" was not recognized and relief was refused those who left WPA jobs. Protest against WPA rates is reported from other communities, including a WPA strike in West Palm Beach, Fla., but as a whole the WPA program begins to settle down and go ahead.

At Chesterfield, St. Louis County, Mo., a second strike was called of 200 workmen employed on U. S. Highway No. 40. The first strike was called in demand for higher wages and second because several issues had not been settled. In the first strike workmen demanded 65 cents an hour instead of 45 cents for laborers and in addition that workers whose pay had been cut be paid 20 cents an hour back pay for the time the men worked at the lower scale. The back pay had not been paid at the time of the second strike. At San Francisco, carpenters in the Bay Counties District seek $1.15 an hour for 7-hour day in place of the present rate of $1.10 for 8 hours. A strike is threatened, while contractors are trying to settle the question through arbitration. The Central California Chapter, Associated General Contractors of America, recently approved a proposal to pay carpenters $1 an hour for 8-hr. day instead of the $7.20 scale generally recognized on private work for the past two years.

The Elmwood Ave. Bridge construction, Rochester, N. Y., was held up on August 29 when 20 riveters walked out in protest because three non-union men were working on the job. One of the three had joined the union but the problem of the other two had not been settled. The bridge is a PWA job and men working on it are hired through the National Re-employment Service.

ENR Cost and Volume Index

The *Engineering News-Record* Construction Cost Index is 195.10 for August, the same value as for July. Minor variations in material costs were purely local in character.

The ENR Volume Index is 140 for August, the highest value since January, 1934, and the second highest value since July, 1933.

INDEX NUMBER

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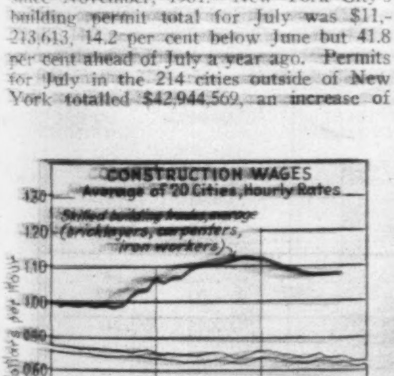

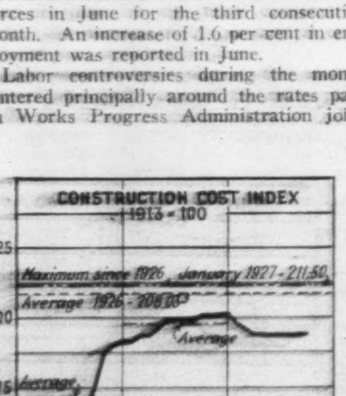

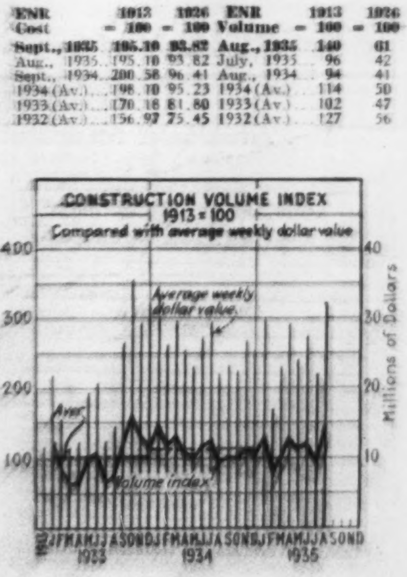

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Bridges, private	.....	909	......	264	25	......	**1,198**	9,416	7,839	.....
Buildings, industrial	720	7,002	982	5,293	2,160	1,554	**17,711**	116,755	69,503	650
Buildings, commercial	1,050	5,537	79	1,014	310	1,228	**9,218**	71,630	60,413	962
Unclassified, private	804	461	420	651	739	994	**4,069**	45,692	28,147	168
Total private	2,574	13,909	1,481	7,222	3,234	3,776	**32,196**	243,493	165,902	1,780
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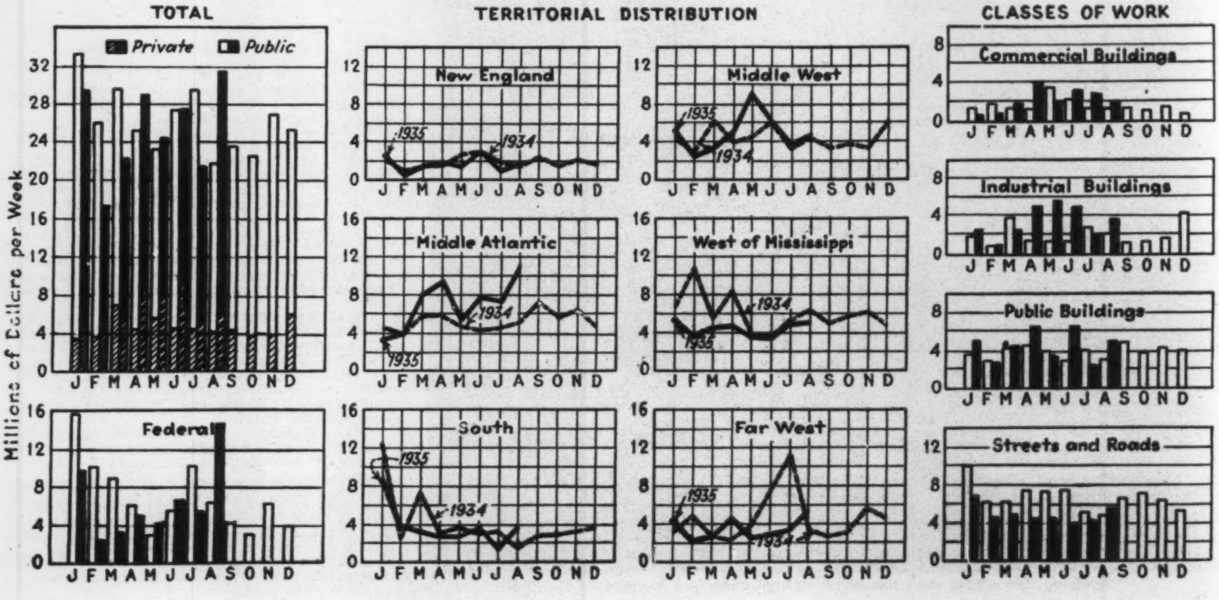
August, 1934 (5 weeks)	7,400	25,003	7,121	21,288	31,712	16,591	**109,115**	......	......	5,509
Eight months, 1935	59,491	247,017	133,874	174,914	158,180	128,725	......	902,201	......	57,442
Eight months, 1934	67,176	170,206	161,429	157,864	218,322	165,092	......	......	940,089	29,606

ENGINEERING CONSTRUCTION REPORTED BY E.N.-R.-WEEKLY AVERAGES

□---1934 ■—1935



preceding month and 22.1 for the corresponding period a year ago according to reports of the American Iron and Steel Institute.

The American Institute of Steel Construction reported for June that shops reporting to them, representing 78 per cent of the industry operated at 38.3 per cent of capacity. June bookings were 60 per cent larger than the average monthly bookings for the first 5 months of this year but about equalled bookings for the same month in 1934. June shipments were approximately 14 per cent less than in 1934. The backlog of the industry was approximately 34 per cent less than the previous year.

During July, production of fabricated steel was 38.5 per cent of normal, according to reports from shops representing 68 per cent of the industry. July shipments were 40.5 per cent of normal compared with an average of 36.6 for the 7 months.

Lumber—Lumber production jumped during the week ended July 27 to the highest total in 16 months while shipments and orders were heaviest since April of this year. Total production of reporting mills was 39 per cent above corresponding week of last year with shipments 46 per cent and new business 13 per cent heavier, according to report of the National Lumber Manufacturers Association.

Lumber consumption for the first half of 1935 was estimated at 8,002,000,000 compared with 7,884,000,000 in 1934. It is expected that total production will be about the same as last year.

Following the record production in the last week of July, each week of the third quarter has seen reported lumber production above the preceding week and with one exception shipments have likewise increased. Shipments during the week ended August 17 were highest of any week of the year except two in April. Production is running about 222,000,000 ft., shipments, 200,000,000 ft., and new business 195,000,000 ft.

Building Permits

The July record of building permits reported by Dun & Bradstreet, Inc., for 215 cities in the United States increased to $54,158,182, a 2.8 per cent gain over June and a 60 per cent increase over July a year ago. This is the largest total reported since November, 1931. New York City's building permit total for July was $11,213,613, 14.2 per cent below June but 41.8 per cent ahead of July a year ago. Permits for July in the 214 cities outside of New York totalled $42,944,569, an increase of 8.4 over last month and 65.2 per cent over last year. By territories the change from last year is: New England, down 42.7 per cent; Middle Atlantic, up 33.1 per cent; South Atlantic, up 102.8; East Central, up 93.5; South Central, up 102.7; West Central, up 199.1; Mountain, up 85.4; and Pacific, up 125.5 per cent.

The Bureau of Labor Statistics, U. S. Department of Labor reports for July a 3 per cent gain in number of permits, but a 1 per cent decrease in indicated expenditures, based on reports from 760 identical cities having a population of 10,000 or more. Comparison with July, 1934, by type of building, indicates an increase this July, based on estimated cost of 214.9 per cent in new residential, 18.4 per cent in new non-residential, and an increase of 35.9 per cent in additions, alterations, repairs.

Prices

Price changes this month are scattered and local. In New York City cement dropped 10c. per barrel, apparently due to the influence of Belgian imports. Sand, gravel and stone dropped 20c. a yard in Los Angeles. Structural clay building tile took a big drop in Cincinnati and a smaller drop in St. Louis. Asphalt prices are lower in Cleveland, and up in Dallas. Roofing is lower in Boston but up in Dallas. Lumber prices in Boston are $1 per M lower on the 1-in. material and $2 per M lower on the 2-in. material, both Southern Pine and Douglas Fir. At Denver and Kansas City, however, Douglas Fir is higher, from $35 to $40 on the 1-in. at Denver and $41 to $44 for 2 x 4-in. and at Kansas City from $24.50 to $27 for 1 x 6-in. and $26.75 to $28 for 2 x 4-in. At both New Orleans and St. Louis, Southern Pine prices are down, $1 per M. at the latter center and various amounts according to size at New Orleans.

Labor

Employment in the production of six major construction materials as reported by the Federal Employment Stabilization Office is 45.5 for July, 1935 compared with 44.6 in June and 43.0 in July, 1934.

According to the Division of Statistics and Information, New York State Department of Labor, lumber and building materials and hardware stores increased their forces in June for the third consecutive month. An increase of 1.6 per cent in employment was reported in June.

Labor controversies during the month centered principally around the rates paid on Works Progress Administration jobs. The New York strike ended when the "strike" was not recognized and relief was refused those who left WPA jobs. Protest against WPA rates is reported from other communities, including a WPA strike in West Palm Beach, Fla., but as a whole the WPA program begins to settle down and go ahead.

At Chesterfield, St. Louis County, Mo., a second strike was called of 200 workmen employed on U. S. Highway No. 40. The first strike was called in demand for higher wages and second because several issues had not been settled. In the first strike workmen demanded 65 cents an hour instead of 45 cents for laborers and in addition that workers whose pay had been cut be paid 20 cents an hour back pay for the time the men worked at the lower scale. The back pay had not been paid at the time of the second strike. At San Francisco, carpenters in the Bay Counties District seek $1.15 an hour for 7-hour day in place of the present rate of $1.10 for 8 hours. A strike is threatened, while contractors are trying to settle the question through arbitration. The Central California Chapter, Associated General Contractors of America, recently approved a proposal to pay carpenters $1 an hour for 8-hr. day instead of the $7.20 scale generally recognized on private work for the past two years.

The Elmwood Ave. Bridge construction, Rochester, N. Y., was held up on August 29 when 20 riveters walked out in protest because three non-union men were working on the job. One of the three had joined the union but the problem of the other two had not been settled. The bridge is a PWA job and men working on it are hired through the National Re-employment Service.

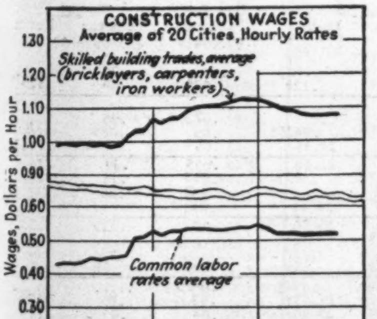
ENR Cost and Volume Index

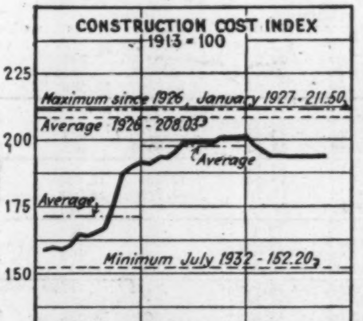
The *Engineering News-Record* Construction Cost Index is 195.10 for August, the same value as for July. Minor variations in material costs were purely local in character.

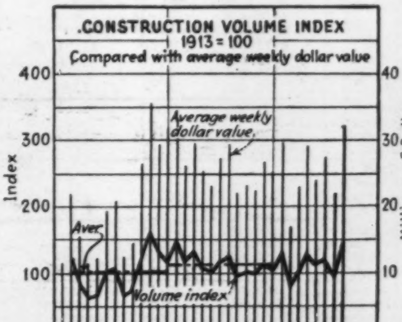
The ENR Volume Index is 140 for August, the highest value since January, 1934, and the second highest value since July, 1933.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Sept., 1935	**195.10**	**93.82**	**Aug., 1935**	**140**	**61**
Aug., 1935	195.10	93.82	July, 1935	96	42
Sept., 1934	200.58	96.41	Aug., 1934	94	41
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56







CURRENT PRICES OF CONSTRUCTION MATERIAL

CEMENT, AGGREGATES AND READY-MIXED CONCRETE—F.O.B. CITY

	PORTLAND CEMENT Per bbl., carload lots, including 40c. per bbl. for bags, cash discount not included			SAND AND GRAVEL Per ton, carload lots			CRUSHED STONE Per ton, carload lots		CRUSHED SLAG Per ton, carload lots, f.o.b. plant		CONCRETE Ready Mixed 1:2:4, 50c.y. or more, delivered
	Cloth Bags*	Paper Bags	Bulk	Gravel, 1½ in.	Gravel, ¾ in.	Sand	1½ in.	¾ in.	1½ in.	¾ in.	
Atlanta	$2.89	$2.64	$2.44	$2.31	$2.31	$1.64	$1.69	$1.92	$1.80	$1.80	$8.25
Baltimore	2.69	2.44	2.24	1.80‡	1.80‡	1.30‡	1.80‡	1.80‡	2.25	2.25	7.35
Birmingham	2.55	2.30	2.10	1.75	1.75	1.40	.85	.85	.90	1.15	5.45
Boston	2.72	2.47	2.27	1.50	1.50	1.10	1.50	1.50	.....	.....	7.00
Chicago	2.85‡	2.60‡	2.40	2.60‡	2.60‡	2.05‡	2.60‡	2.60‡	.....	.....	.....
Cincinnati	2.60	2.35	2.15	1.20	1.20	1.10	1.80	1.80	.....	.....	6.70
Cleveland	2.41	2.16	.....	1.55	1.55	1.45	1.60	1.60	1.75	1.75	8.85
Dallas	2.45	2.20	2.00	1.55	1.55	1.30	2.10	2.25	.....	.....	7.00
Denver	3.09	2.84	.....	1.85	.....	1.25	.....	.....	.....	.....	.....
Detroit	2.26	2.01	1.81	1.30	1.30	1.20	1.40	1.60	1.40‡	1.50‡	6.75
Kansas City	2.50	2.25	2.05	.....	1.85	1.15	1.85	1.40-1.55	.....	.....	7.65
Los Angeles	2.70	2.42	.....	1.40	1.40	1.00[s]	1.40	1.40	.....	.....	6.70‡
Minneapolis	2.90	.....	2.45	1.00[p]	1.00[p]	.25[p]	1.00[p]	1.00 [p]	.....	.....	5.75
Montreal	.....	1.70	.....	.85	.90	1.25	1.25	1.60	.....	.....	6.00
New Orleans	2.40	2.22	2.00	1.75	1.75	1.45	.....	.....	.....	.....	8.25
New York	2.50‡	2.25‡	.....	1.25†[b]	1.25†[b]	.75†[b]	1.75†[b]	1.85†[b]	.....	.....	7.60
Philadelphia	2.63	2.38	2.18	1.80‡	1.90‡	1.50‡	2.60‡	2.75‡	1.00‡	1.00‡	7.75
Pittsburgh	2.46	2.21	1.61	1.50‡	1.50‡	1.70‡	2.25‡	2.25‡	1.50	1.50	7.00
St. Louis	2.53	2.28	.....	1.00‡	1.00‡	1.00	.75[p]	.75[p]	.75§	.75§	6.65
San Francisco	2.72	2.52	2.20	1.35	1.35	1.35	1.50	1.50	.....	.....	7.53
Seattle	3.09	2.84	.....	1.00	1.00	1.00	1.44	1.44	.....	.....	7.56

*10c. allowed for each returnable bag. †Per cu. yd. ‡Delivered. §F.o.b. Granite City, Ill. [b]Barge lots alongside dock. [p]f.o.b. plant. [s]5 tons or more.

CURRENT MAXIMUM NET PRICES RECEIVED AT CEMENT MILL

Charge for bags not included. For cloth bags, add 40c. per bbl.; 10c. refund allowed for each returnable bag; for paper bags add 15c. per bbl., not refundable.

	Bagged	Bulk		Bagged	Bulk		Bagged	Bulk
Alpena, Mich.	$1.50	$1.45	Ironton, Ohio	$1.60	$1.55	North Birmingham, Ala.	$1.75	$1.70
Buffington, Ind.	1.70	1.65	Iola, Kans.	1.70	1.65	Richard City, Tenn.	2.00	1.95
Crestmore, Calif.	1.72	....	La Salle, Ill.	1.60	1.55	Saginaw, Mich.	1.50	1.45
Dallas, Tex. (Inc. 5c. tax)	1.80	1.75	Limedale, Ind.	1.70	1.65	Steelton, Minn.	1.80	1.75
Hannibal, Mo.	1.70	1.65	Mason City, Ia.	1.70	1.65	Universal, Pa.	1.65	1.60
Hudson, N. Y.	1.75	1.70	Norfolk, Va.	1.76	1.71	Waco, Tex. (Inc. 5c. Tax)	1.80	1.75
Independence, Kans.	1.70	1.65	Northampton, Pa.	1.65	1.60	Wyandotte, Mich.	1.45	1.40

Cash Discounts, Cement, to Contractors: truck delivery, 2% for cash on 10th of month; on carload deliveries, no trucking, same as dealer cash discount, 10c. per bbl. for payment within 15 days of date of invoice.

STRUCTURAL CLAY BUILDING TILE, BRICK AND LIME—DELIVERED

	STRUCTURAL CLAY TILE—PARTITION—SCORED Per M, lots of 2,000 pieces or over,			STRUCTURAL CLAY TILE—LOAD BEARING—SCORED Per M, lots of 2,000 pieces or over,			BRICK Per M, in quantity,		LIME Per ton, in paper, carload lot,		
	3x12x12 in.	4x12x12 in.	8x12x12 in.	8x12x12 in.	10x12x12 in.	12x12x12 in.	Common backing	Straight hard	Hydrated finishing	Common hydrated	Pulverized or lump
Atlanta	$75.80	$80.80	$151.60	$181.90	$212.20	$242.50	$14.25	$14.25	$24.00	$17.00	$15.90
Baltimore	91.00	98.00	183.00	219.00	315.00	390.00	13.00	17.00	17.00	12.50	19.25
Birmingham	78.75	84.00	157.50	189.00	201.25	242.00	15.00	22.50	3.00§	1.75§	2.25§[e]
Boston	72.50[s]	77.45[s]	145.35[s]	164.50[s]	203.70[s]	......	14.50	18.00	17.50	12.50	3.10[g][e]
Chicago	66.00	70.40	132.00	......	......	......	10.20	11.20	18.00	13.40	14.00
Cincinnati	55.00	58.70	110.00	124.10	164.20	187.70	17.50	17.50	15.97	12.48	.....
Cleveland	45.00	48.00	90.00	108.00	188.00	214.75	16.50	16.50	16.00k	14.00k	2.65k[p]
Dallas	72.00	75.00	197.00	177.00	217.00	269.00	13.50	26.00	21.20	14.50	1.70 [p]
Denver	78.50	88.00	141.50	166.50	196.00	225.00	10.00–15.50	21.00–30.00	25.00	17.00	20.00
Detroit	71.85	76.65	143.70	230.00	280.00	325.00	11.50	12.50	14.25	11.90	22.20
Kansas City	72.50	72.50	92.00*	140.50	177.75	207.25	15.00	24.00	15.00	11.25	14.00[e]
Los Angeles	77.50	94.25	134.00*	188.00	232.50	311.50	11.50	14.00	18.70	.....	19.70
Minneapolis	80.15[s]	85.50[s]	145.40[s]	167.75[s]	175.25[s]	243.70[s]	13.10	15.10x	25.50	21.00	23.00
Montreal	79.10[s]	90.40	169.50[s]	......	212.40	136.60	20.75	31.00	23.00	11.00	.....
New Orleans	61.60	65.10	123.20	158.90	196.70	......	14.00	.....	16.75	13.05	.....
New York	84.00	90.00	135.00*	193.30†	238.80†	295.60†	10.50	.....	20.80	16.00	20.80
Philadelphia	76.00	80.00	150.00	172.00	240.00	295.00	18.00	20.00	16.35	11.25	10.75
Pittsburgh	58.40c	62.30c	116.85c	165.00c	217.00c	250.00c	15.00	22.00	17.20	14.50	2.22[p]
St. Louis	58.00	60.00	120.00	185.00	180.00	210.00	15.00	18.00	.48‡	.38‡	2.80[p]
San Francisco	84.00	94.50	225.00	......	......	......	15.00	18.00	22.50	21.50	2.05[p]
Seattle	86.00	94.50	210.00	198.00	198.00	198.00	15.00[d]	18.50	30.00	26.00	3.00[p]

*6x12x12 in. †F.o.b. Perth Amboy, N. J. ‡Per sack. §Per bbl., 200 lbs. [p]Per bbl. 180 lbs. [g]per bbl. 280-lb. kLCL. [d]less $1 cash 15 days. [e]lump. [s]Smooth. x Selected common.

ROAD SURFACING MATERIALS—F.O.B. CITY

	PAVING BRICK AND BLOCKS			PAVING ASPHALT Per ton, less than 80 penetration,		ASPHALT BINDERS—FLUXES Per gal., 80-300 penetration,		CUTBACK ASPHALT Per ton,		ROAD OILS Per gal.,	ASPHALT EMULSION Per gal.,	
	Granite per M, lots of 50,000, 4x4x8 in.,	Brick per M, 3x8½x4 in., carload lots	Wood per sq.yd., 3½ in., 16-lb. treat,	Tank car	Drums	Tank car	Drums	Tank car	Drums	Tank car	Tank car	Drum
Atlanta	$ 75.00	$35.00	$2.25	$17.40[2]	$22.57[2]	$0.0644[2]	$0.911[2]	$15.90[2]	$23.82[2]	$0.0586[2]	$0.085	$0.085
Baltimore	110.00	44.00	2.35	17.00	22.00	.07	.095	.085†	.12†	.....	.....	.....
Birmingham	110.00	24.00	....	18.00	23.00	0.778	.1028	.082†	.112†	.....	.06	.....
Boston	78.00	30.00	2.40	16.00	21.00	16.50#	21.50#	.09†	.12†	.075	.09	.125
Chicago	125.00	42.00	2.60	.....	.....	....	.....	.....	.....	.....	.....	.....
Cincinnati	115.00	37.00	....	18.00	22.00	.09	.....	18.50	.....	.07	.08	.....
Cleveland	100.00‡	32.75	2.85	19.00	23.25	.075	.085	.075†	.0425†	.06	.085	.10
Dallas	.....	30.00	....	17.30	22.30	.0575	.105	16.85	24.35	.06	.1126	.1488
Detroit	.....	37.00	....	19.67[a]	24.87[a]	.....	.....	17.00	22.50	.055	.11	.15
Kansas City	.....	45.00	....	18.50	23.50	17.50#	22.50#	.077†	.107†	.057	.11	.15
Los Angeles	.....	50.00	....	10.34	14.50	10.34#	14.50#	10.34	.....	.042	.06	.12
Minneapolis	.....	.....	2.50	18.20	24.70	.098	.098	21.85	34.65	.048	.....	.....
Montreal	70.00	100.00	....	14.00	19.66	.07	.093	19.00	26.50	.07	.125	.155
New Orleans	100.00	32.00	....	14.50	18.50	.065	.09	.07†	.10†	.....	.....	.....
New York	130.00§	50.00	2.64	17.00	22.00	.07	.095	.085†	.12†	.....	.075	.11
Philadelphia	122.00	45.00	2.25	13.50	18.00	.05	.08	.05†	.08†	.045	.08	.....
Pittsburgh	115.00	40.00	....	19.00	24.00	.0775	.1125	.086†	.121†	.0675	.....	.....
St. Louis	115.00	35.00	2.00	19.70	24.70	.075	.075	.875†	11.75†	.04	.09	.125
San Francisco	.....	50.00	....	12.00	18.00	12.00#	18.00#	12.50#	23.00#	.04	.0575	.117
Seattle	.....	55.00	....	25.50	25.50	17.00#	23.00#	15.00	.....	.03	.19	.20

Note: Paving asphalt, tank car or boat, f.o.b. Maurer, N. J., per ton, Bermudez, $25.00; Trinidad, $21.00. *Per sq. yd. †Per gallon. ‡4½x6x5½in. #Per ton §4x5x9-in. Av. [a]Mexican. [2]local reduction due to 20% reduction intra-state class freight rates, only Georgia affected.

CURRENT PRICES OF CONSTRUCTION MATERIALS

IRON AND STEEL PRODUCTS—BASE MILL PRICES

	STRUCT. SHAPES-PLATE	REINF. BARS ¾-in. billet,	RIVETS ¾-in. structural	WIRE NAILS Base	SHEET PILING Base	STEEL RAILS Per Gross Ton Standard	Light	Re-rolled	TRACK SUPPLIES Angle bars	Std. spikes	Tie plates	Track bolts
Birmingham...	$1.95	$2.10	.....	.....	.....	$36.375	$34.375	$35.00	$2.55	$2.40	$1.90	$3.55
Chicago........	1.85	2.10	$3.00	.....	$2.25	36.375	35.00	34.00	2.55	2.40	1.90	3.55
Pittsburgh.....	1.80	2.05	2.90	$2.40	2.15	36.375	35.00	34.00	2.55	2.40	1.90	3.55

IRON AND STEEL PRODUCTS—F.O.B. WAREHOUSE, PER 100 LB., BASE PRICE

F.O.B. WAREHOUSE	STRUCTURAL SHAPES Per 100 lb., base price	REINFORCING BARS Per 100 lb., ¾ in., base price New billet	Rail steel	EXPANDED METAL LATH —Per 100 sq.yd., carload lots— Std. diamond mesh, 3.4 lb.	Std. ribbed 3.4 lb.	WELDED FABRIC REINFORCING —Per 100 s.f., carload lots— 4x16 in., No. 5 & 10 wires	4x12 in., No. 8 & 12 wires	6x6 in., No 6 & 6 wires Per sq.yd	SHEET PILING Per 100 lb., base price
Atlanta...............	$3.36	$2.34	$2.19	$21.00	$23.00	$1.53	$1.16	$0.1449	$2.70
Baltimore..............	3.00	2.75	2.60	25.00	28.00	1.41	1.08	.1332	2.90
Boston...............	3.54	3.18	2.25	21.50†	23.50†	1.46	1.12	.1449	2.30
Chicago..............	3.20	2.10	1.95	22.00	26.00	1.40	1.08	.1323	2.25
Cincinnati............	3.40	3.25	3.25	25.01	29.50	1.39	1.07	.1287	....
Cleveland.............	3.16	2.60	2.45	20.00	22.00	1.37	1.06	.1287	....
Dallas...............	2.225	2.45	2.30	22.00	25.00	1.63	1.23	.1548	2.985
Denver...............	3.95	3.76	3.61	22.00	25.00	1.68	1.26	.1548	....
Detroit...............	3.17	2.20	2.05	20.00†	22.00†	1.41	1.08	.1323	2.43
Kansas City...........	3.74	2.74	2.59	19.00	21.00	1.50	1.14	.1422	2.69
Los Angeles...........	3.10	2.45	....	23.00	27.50	1.68	1.26	.1584	2.65
Minneapolis..........	3.45	2.86	2.71	22.00	26.00	1.51	1.15	.1431	2.55§
Montreal..............	2.90	3.00	2.80	.....	.....	2.30	1.71	.238	2.65
New Orleans..........	....	....	....	19.15	23.00	1.54	1.17	.1458	....
New York.............	3.37	2.86	2.71	20.00	22.00	1.44	1.11	.1359	2.50
Philadelphia..........	2.98	3.06	2.22	20.00	24.00	1.42	1.09	.1332	2.46
Pittsburgh.............	1.80‡	1.80‡	1.70‡	20.00‡	22.00‡	1.34	1.04	.1260	2.15
St. Louis.............	3.45	2.95	2.80	22.00	24.00	1.42	1.09	.1341	2.25
San Francisco.........	3.55	2.475	2.325	23.00	28.00	1.68	1.26	.1584	2.60
Seattle...............	3.50	2.50	....	30.00	33.00	1.68	1.26	.1584	2.60

‡Base Mill Price. Note: Shapes, bars and piling subject to quantity discounts. §Mill Price plus freight to Minneapolis. †f.o.b. delivered.

PAINTING AND ROOFING SUPPLIES—F.O.B.

	RED LEAD Per 100 lb. in 500-lb. lots Dry	WHITE LEAD Per 100 lb. in 500-lb. lots# in oil	READY-MIXED PAINT Per gal., drums Graphite*	Aluminum†	Ferric Oxide‡	ROOFING SUPPLIES Carload lots, f.o.b. factory Rolls, slate surfaced, 85-90 lb., per sq.	Asphalt felt, per 100 lb.	Tar felt, per 100 lb.	Asphalt coating, per gal.	Tar pitch, 350-lb. bbl., per ton
Atlanta..........	$7.75	$10.50	$1.60	$3.10	.....	$1.81	$1.97	$2.90	•.36	$25.40
Baltimore........	7.00	10.00	1.40	2.25	$1.90	2.35	2.55	2.55	.48	25.00
Birmingham......	7.25	10.50	2.35	2.35	1.85	2.20	1.79	....	.36	23.00
Boston..........	7.00	10.00	1.80	2.65	.04½x	1.80	2.80	2.80	.40	20.00
Chicago..........	7.00	10.00	1.85	2.45	1.30	1.60	1.59[a]	1.59[a]	.24	20.00
Cincinnati.......	7.00	10.00	1.45	2.50	1.40	2.40	2.39[a]	2.39[a]	.36	21.00
Cleveland........	7.00	10.00	1.65	2.30	1.30	1.60	1.79	1.79[a]	.24	22.00
Dallas....	7.25	11.00	1.80	7.24	.....	2.60[a]	4.47	4.47	.16	37.50
Denver	7.50	10.75	2.55	.....	.....	2.75	3.46	3.46	.60	30.00
Detroit..........	7.00	10.00	1.55	2.20	1.30	2.55	2.80	2.55	.35	30.00
Kansas City......	7.00	10.12½	1.25	2.25	1.00	2.05	3.10–3.20	3.10–3.20	.23	26.50
Los Angeles..	7.25	10.25	1.60	2.25	2.30	2.00	1.50	.....	.45	.....
Minneapolis.....	7.00	10.12½	.....	.....	.....	1.60	1.79[a]	1.79[a]	.29	25.30
Montreal........	....	8.20	1.70	3.45	1.65	2.00	2.62	2.02	1.40§	1.52§
New York.......	7.00	10.00	1.60	2.00	1.40	1.60	1.79[a]	1.79[a]	.232	22.00
Philadelphia...	7.00	10.00	2.50	2.95	1.50	1.68	1.86[a]	1.48[a]	.37	22.00
Pittsburgh.......	7.00	10.00	1.05	1.95	.80	1.75	1.65[a]	1.65	.38	25.00
St. Louis.........	10.00	10.00	1.85	2.45	1.30	1.87[a]	3.22	3.22	.28	20.00
San Francisco....	7.25	10.25	1.80	2.75	1.10–2.00	1.89	3.10	3.10	.38	26.00
Seattle..........	7.25	10.25	1.80	2.85	2.20	3.30	2.66	2.60	.60	30.00

*U. S. War Dept. Spec. 3-49A. †ASTM Spec. D266-31. ‡80% maximum ferric oxide. §Per 100 lb. [a]Per roll. 65 lb. xPer lb. •500-lb. drums.
Note: Red lead in oil 50c higher than white lead in oil. #Basis of quoting changed this month from list to 500-lb. lots.

MISCELLANEOUS ITEMS

	WINDOW GLASS Discounts from latest jobbers List, Sept. 15, 1928 Single or Double Thickness A quality	B quality	EXPLOSIVES Per lb. dynamite delivered 50-lb. cases Gelatin 40%	60%
Atlanta............	85%	85%	$0.155	$0.17
Baltimore...........	88–10%	90–10%	.16	.175
Birmingham.........	88%	90%	.105	.120
Boston.............	88–10–5%	90–10–5%	.185	.195
Chicago............	90%	92%	....	....
Cincinnati..........	89–10%	91–10%	.27	.29
Cleveland...........	89-10%	91-10%	.19	.205
Dallas..............	86%	88%	.17	.185
Denver..............	86%	88%	.155	.17
Detroit.............	87–10%	89–10%	.237	.255
Kansas City.........	87–10%	89–10%	.16	.175
Los Angeles.........	90%*	90–40%	.1725†	.1925†
Minneapolis.........	86%	88%	.175	.19
Montreal............	......	......	.1875	.2125
New Orleans........	88%	90%	.19	.22
New York...........	88%	90%	.225	.24
Philadelphia........	88%	90%	.1550	.17
Pittsburgh..........	88%	88%	.11	.1250
St. Louis...........	90%	92%	.135	.15
San Francisco.......	82%	87%	.1575	.1775
Seattle.............	90–10%	90–20%	.1425	.1575

*Disc. from list Aug. 1, 1929. †F.o.b. Arsenal ‡Double thickness A & B

CHEMICALS

Water, sewage treatment, road work, f.o.b. carlots, New York

Bleaching powder, in drums, f.o.b. works, per 100 lb.	$1.90@$2.15
Calcium chloride, 77-80%, flaked, in 400-lb. drums or 100-lb. moisture proof bags, f.o.b. works, freight equalized with points of competition, per ton	22.00@35.00
Chlorine cylinders, per lb. delivered	.055
Silicate of soda, 52 deg., in drums, f.o.b. works, per 100 lb.	1.35
Soda ash, 58%, in paper bags, per 100 lb.	1.23
Sulphate of aluminum, commercial, in 100 lb. bags per ton	27.00
Sulphate of copper, in bbl., per 100 lb.	3.85

PILES

Prices per linear foot, pine, with bark on f.o.b., New York; delivered from barge, 1½ to 2c. per ft. additional:

Dimensions	Points	Length	Short Leaf Barge	Rail	Long Leaf Barge	Rail
12 in. at butt.........	6 in.	30 to 50 ft.	$0.12	$0.165	$0.13	$0.16
12 in.—2 ft. from butt	6 in.	50 to 59 ft.	.16	.21	.15	.20
12 in.—2 ft. from butt.	6 in.	60 to 69 ft.	.17	.225	.16	.21½
14 in.—2 ft. from butt.	6 in.	50 to 69 ft.	.195	.235	.20½	.24½
14 in.—2 ft. from butt.	6 in.	70 to 79 ft.	.2125	.245	.24½	.30½
14 in.—2 ft. from butt.	5 in.	80 to 85 ft.	.275	.385	.....	......
14 in.—2 ft. from butt.	5 in.	85 to 89 ft	.315	.452	.....	......

RAILWAY TIES

Prices f.o.b., per tie, for carload lots:		6 In. x 8 In. by 8 Ft.	7 In. x 9 In. by 8½ Ft.
Boston.......	Oak, untreated	$1.20	$1.75
	Pine, untreated	1.25	1.75
	Pine, creosoted	1.70	2.40
New York....	Long-leaf sap pine, untreated	.80	1.00
	Mixed oak, untreated	....	1.00
Birmingham .	White oak, untreated	.75	1.50
	White oak, creosoted	no market	no market
	Southern pine, untreated *6x8x8'6'	.45*	.60
	Southern pine, creosoted	1.00*	1.20
Chicago......	White oak, untreated	1.15	1.40
	Oak, empty cell, creosoted	1.50	1.88
	Oak, zinc treated	1.40	1.70
	Southern pine, creosoted	1.35	1.78
Los Angeles..	Fir, untreated	.50	.72
	Fir, creosoted	1.67	2.40
Philadelphia..	Red oak, untreated	1.00	....
	Red oak, creosoted	....	2.10
St. Louis.....	White oak, untreated	1.03	1.35
	Red oak, untreated	1.00	1.30
	Red oak, creosoted	1.45	1.90
San Francisco	Sap Pine or cypress, untreated	.80	1.20
	Douglas fir, green, untreated	.45	.63
	Douglas fir, empty cell, creosoted.	1.14	1.56
Montreal	Birch or maple, untreated	1.00	1.10
	Birch or maple, creosoted	1.25	1.55

CURRENT PRICES OF CONSTRUCTION MATERIALS

WATER, SEWER AND DRAIN PIPE

	C. I. PIPE	**VITRIFIED SEWER PIPE**				**CLAY DRAIN TILE**		**CONCRETE SEWER PIPE**		**WROUGHT STEEL PIPE**			
	Per net ton, f.o.b. 6 in. to 24 in.*	Per foot, delivered, standard (‡ f.o.b.)				Per 1,000 ft., carload lots, f.o.b.		Per ft., delivered; ASTM C 14-24		Full standard weight. #			
										1 to 3 in., Butt Weld		3½ to 6 in., Lap Weld	
		8 in., s.s.	12 in., s.s.	24 in., d.s.	36 in., d.s.	6 in.	8 in.	12 in.	24 in.	Black	Galv.	Black	Galv.
Atlanta	$43.00	$0.225	$0.405	$1.4625	$3.25	$80.00	$128.00	$0.50	$1.625	44%	37%	42%	34%
Baltimore	46.50	.25	.50	2.10	5.75	108.00	168.50	.75	2.00	.....	.....	.....	.....
Birmingham	39.00	.25	.45	1.625	4.66	85.00	200.00	.45	1.60	48.939%	37.918%	50.162%	39.143%
Boston	46.50	.196	.378	1.72	5.06	51.10	85.85	.378	2.15	.....	.....	.....	.....
Chicago	47.50	.285	.54	1.98	5.60	85.00	120.00	.675	2.25	56%	45%	55%	46%
Cincinnati	46.00	.203	.3915	1.665	5.125	69.00	108.00	.40	1.40	53.04%	42.02%	54.265%	43.245%
Cleveland	47.50	.196	.378	1.62	5.0225	47.25	72.00	.536	2.102	58.4%	48.4%	60.6%	49.6%
Dallas	51.00	.25	.45	1.80	.....	160.00	250.00	.41	1.41	64%	55%	64%	56%
Denver	59.00	.23‡	.415‡	1.73‡	.....	.....	.....	.....	.....	56%	45%	43%[b]	57%[b]
Detroit	47.50	.245	.4725	2.025	5.74	82.00	136.00	.35	1.20	48.6%	38.2%	47.3%	37.6%
Kansas City	49.00	.32	.57	2.27	4.90	120.00	240.00	.40	1.50	56%	47%	57%	48%
Los Angeles	47.50	.2825‡	.508‡	2.03‡	5.535‡	115.50	192.50	1.00	2.00	53.57%	43%	53%	42.51%
Minneapolis	49.50	.255	.459	1.836	4.67	.....	.....	.50	1.34	....	....	....	....
Montreal	55.00	.80	1.50	6.00	.....	45.00	90.00	.45	.....	....	....	....	....
New Orleans	45.00	.33	.52	1.65	.....	67.00	......	.45	2.00	....	....	....	....
New York	46.00	.26	.51	2.25	6.75	.....	......	.55	2.00	51.88%[a]	40.86%[a]	53.10%[a]	42.08%[a]
Philadelphia	43.00	.336	.648	2.835	8.68	175.00	210.00	.54	1.80	52.37%[a]	41.35%[a]	53.39%[a]	42.57%[a]
Pittsburgh	49.00	.175‡	.3375‡	1.44‡	4.81‡	69.00	108.00	.75c	1.75	64%	55%	65%	56%
St. Louis	46.00	.215	.459	1.836	3.96	85.00	170.00	.80	2.00	51.266%	40.245%	52.49%	41.47%
San Francisco	47.00	.265	.477	1.908	5.22	103.50	172.50	.45	2.20	52.4%	63.4%	51.1%	64.6%
Seattle	48.50	.35†	.63†	2.52†	6.30†	96.00	200.00†	.50†	1.80	60%	50%	65%	50%

*b. & s. class B and heavier, C/L lots, 200 tons and over. Burlington, N. J. (base) $43.00. Gas pipe and class A, $3 per ton additional, 4-in., $3 per ton additional. 30 in. and larger usually $2 per ton less. ‡Price at factory. †Less 30% in C/L Lots.

#Discounts from standard list for delivery from warehouse, except Pittsburgh prices are f.o.b. mill. Base price $200 per net ton. List prices per ft.: ½ in., 8½c; ¾ in. 11½c.; 1 in., 17c.; 2 in., 37c.; 2½ in., 58½c.; 3 in., 76½c.; 4 in., $1.09; 6 in., $1.92.

[b]2½ to 3 in. [a]Resale price to consumer, car load lots shipped from mill.

c Concrete Culvert pipe reinforced.

LUMBER AND TIMBER—PER M FT. B.M., CARLOAD LOTS F.O.B.

		SHORT LEAF YELLOW PINE AND DOUGLAS FIR									**LONG LEAF Y. P.**	
		All S. L. Y. P. is No. 2 common or better and for No. 1 N. C. Box. All Fir planks No. 2 common; Fir timber is No. 1 common. Lengths up to 20 ft.									Merchantable grade up to 20 ft.	
		1x6,^{S}	1x8,^{S}	2x4,^{S}	2x6,^{S}	2x8,^{S}	2x10,^{S}	3x12,^{R}	6x12,^{R}	12x12,^{R}	2x12,^{R}	12x12,^{R}
Atlanta	Southern Pine	$16.00	$16.00	$18.00	$18.50	$20.00	$19.50	$28.00	.....	.....	$38.50	$40.00
Baltimore	Southern Pine	20.50	24.00	29.50	28.50	29.00	30.50	37.50	.....	.....	54.00	55.00
	Douglas Fir	29.50	29.00	31.00	29.00	30.00	30.50	32.00	$33.00	$34.00	.....	.....
Birmingham	Long Leaf	17.00	17.25	17.25	15.25	16.25	16.25	38.00	.....	.....	33.00	35.80
	Southern Pine	17.00	17.25	16.00	14.00	15.00	15.00	25.00	25.00	24.00	.....	.....
Boston	Southern Pine	27.00	27.00	33.00†	35.00†	51.00‡	60.00‡	73.00‡	.....	.....	73.00	73.00
	Douglas Fir	27.00	27.00	33.00	33.00	33.00	35.00	40.00	43.00	43.00	.....	.....
Chicago	Southern Pine	27.00	28.00	33.50	30.00	31.00	32.50	45.00	.....	.....	53.00	57.00
	Douglas Fir	28.50	29.50	31.75	30.50	30.50	28.50	40.75	41.75	41.75	.....	.....
Cincinnati	Southern Pine	24.00	25.00	26.00	25.00	25.50	26.50	40.00	.....	.....	31.50	.....
	Douglas Fir	.....	.....	.....	.....	.....	.....	43.50	46.25	46.25	.....	.....
Cleveland	Southern Pine	26.00	28.00	32.00	28.00	30.00	32.00	45.00	43.00	.....	48.00	58.00§
Dallas	Southern Pine	d31.50	32.40	35.55	32.50	35.50	35.50	55.00	.....	.....	51.90	64.35
Denver	Douglas Fir	40.00	41.00*=	44.00*	44.00*	44.00*	46.00*	52.00*	56.00*	61.50	.....	.....
Detroit	Southern Pine	24.00	24.00	25.00	24.50	24.50	25.50	35.00	.....	.....	29.00	41.50
	Douglas Fir	.....	.....	.....	.....	.....	.....	41.50	41.50	41.50	.....	.....
Kansas City	Southern Pine	24.50	25.50	24.50	24.50	26.00	28.00	35.00	.....	.....	54.50	60.50
	Douglas Fir	27.00	28.00	28.00	27.00	27.00	30.00	35.00	37.00	37.00	.....	.....
Los Angeles	Douglas Fir	24.00y	25.50y	26.00y	26.00y	26.00y	26.00y	38.00y	30.00y	34.00y	.....	.....
Minneapolis ^{c}	Western Pine	45.00*	51.50*	50.00*	41.00*	41.00*	42.00*	59.50*^{b}	.....	.....	58.00*^{b}	.....
	Douglas Fir	39.00*	42.00*	43.50*	41.00*	41.00*	42.00*	59.50*	59.50*	59.50*	.....	.....
Montreal	Spruce	30.00	31.00	27.00	29.00	32.50	35.00	38.00	.....	.....	.....	.....
	Douglas Fir	47.50	53.00	42.50	44.00	44.00	45.75	51.00	51.00	51.00	43.00	.....
New Orleans	Southern Pine	23.90	25.90	29.00	25.75	26.75	26.00	39.00	.....	.....	45.00	55.00
	Douglas Fir	28.95	31.50	33.90	29.35	32.90	33.95	37.95	.....	.....	.....	.....
New York	Southern Pine	34.00*	35.00*	34.00*	35.00*	36.00*	37.00*	45.00*	.....	.....	70.00*	70.00*
	Douglas Fir	.....	.....	38.00*	37.00*	38.00*	39.00*	38.00*	42.00*	42.00*	.....	.....
Philadelphia	Southern Pine	30.00	31.00	32.00	32.00	33.00	34.00	36.00	.....	.....	36.00	42.00
	Douglas Fir	45.00	45.00	45.00	45.00	45.00	45.00	45.00	42.00	42.00	.....	.....
Pittsburgh	Southern Pine	30.50	31.90	37.65	33.75	36.52	37.50	47.75	.....	.....	57.50	57.50
	Douglas Fir	38.52	39.70	39.70	38.55	39.10	39.38	53.25	53.25	53.25	.....	.....
St. Louis	Southern Pine	28.00*x	29.00*x	29.00*x	28.00*x	29.00*x	29.50*x	48.00*x	.....	.....	.....	.....
	Douglas Fir	.....	.....	.....	.....	.....	.....	.....	54.00*x	54.00*x	71.00	69.00
San Francisco	Douglas Fir	20.00t	20.00t	21.50t	21.50t	22.00t	22.50t	24.50	25.00t	25.00t	.....	.....
Seattle	Douglas Fir	14.00	15.00	15.00	14.00	14.00	14.00	20.00	19.00	19.00	.....	.....

*Delivered. †Spruce. ‡Long-leafed Y. P. §Norway Pine. ^{b}Northern Pine. =Native. t At ship's tackle. y Yard prices. d 10% Disc. taken off. x 5M ft. or less. ^{S}S4S. ^{R}Rgh. ^{c}Contractors in Minneapolis and St. Paul get 20% disc. from list.

CURRENT BUILDING AND CONSTRUCTION WAGE RATES PER HOUR

	Bricklayers	Carpenters	Structural Iron Workers	Hoisting Engineers	Plasterers	Common Labor	
						Building	Heavy Const.
Atlanta	$0.70/1.125	$0.40/.90	$0.60/1.25	$0.60/1.25	$0.60/1.00	$0.30/.40	$0.30/.40
Baltimore	1.25	1.10	1.37½	1.25	1.25	.45	.45
Birmingham	1.00	.60/1.00	1.00	1.125	1.00	.30/.40†	.30/.40†
Boston	1.20	.70/1.17½	1.20	.90/1.37½	1.37½	.50/.70	.50/.70
Cincinnati	.80/1.375	.65/1.20	1.25	1.00/1.25	.90/1.375	.45	.45
Chicago	1.50	1.31¼	1.35	1.31¼	1.50	.82½	.82½
Cleveland	1.25	1.12½	1.25	1.125	1.375	.72	.72
Dallas	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.25/.40	.25/.40
Denver	.60/1.12½	.60/.87½	.60/.87½	.60/.87½	......	.50	.50
Detroit	1.25	1.00	1.25	.75/.95	1.25	.55/.60	.55/.75
Kansas City	1.32½	1.12½	1.125	1.12½	1.32½	.70	.45/.50
Los Angeles	1.10	1.10	1.25	1.25	1.25	.50	.60
Minneapolis	1.00	.75	1.20	.75	.90	.45	.45/.50
New Orleans	1.00	.75	1.00	1.00	1.25	.30/.40	.30/.40
New York	1.50	1.40	1.65	1.50/1.65	1.50	.82½	.50/.80
Philadelphia	.35/1.50	.65/1.10	.70/1.375	.90/1.00	1.10/1.37	.35/50	.40/.50
Pittsburgh	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.35/.50	.35/.50
St. Louis	1.50	1.25	1.47	1.35/1.47	1.50	.78¾	.35/.78
San Francisco	1.50*	1.10	1.37½	1.12½	1.25*	.62½	.60
Seattle	1.00	1.00	1.25	1.25	1.25	.60	.60
Montreal	1.00	1.00	1.25	1.25	1.25	.60	.60

Skilled Avg: (Bricklayers, Carpenters, Ironworkers) $1.084 Common Avg. $0.5285 *6-hr. Day. † PWA.